Applied Seismology for Engineers

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Week – 09 Lecture - 01

Lecture – 21

Hello everyone, welcome to lecture 21 of the course Applied Seismology for Engineers, myself, Dr. Abhishek Kumar. In the discussion so far, we have discussed about the process of earthquake occurrence, starting from the source, how the waves which will be generated at the source will undergo attenuation, loss because of heat, particle oscillation, scattering. So, subsequently, these waves, after a lot of changes, will reach to the bedrock at the site of interest and then subsequently modifying the characteristics of these vibrations which generated from the source, subsequently modified by the propagation path and then subsequently, it will be modified by different layers which are available above the bedrock level and up till the surface level. As a result, there will be significant change in the ground motion characteristics between the bedrock, between the surface. In addition, when we were discussing about different kinds of waves, we also discussed, thus as the wave propagates through a particular medium primarily, I am talking about seismic waves or specifically about P wave, S waves. So, when waves are propagating through a particular medium, this will cause either compression, rarefaction or shearing in the medium.

So, in today's class, we will be discussing about a critical phenomenon, which is called as liquefaction. As the name suggests, it is basically related to the transformation of the soil which during static condition, in general, is offering lot of resistance, lot of bearing capacity. As far as the same soil, primarily witnessed in cohesionless soils, subjected to high groundwater table. So, when such a condition is exposed to earthquake loading, primarily moderate to strong earthquake loadings, what will happen that because of the propagation of the waves, there will be development of excess pore pressure. So, even though there was some pore pressure, but as the wave pass through a particular medium, which you can see wave propagating from bedrock to the surface, when these waves are subjected to a particular medium, it will trigger additional loading on your soil medium. As a result of this additional loading, there will be development of excess pore pressure. Whatever pore pressure was there, because of the presence of water in the soil medium, now that will start increasing because of additional loading. Primarily, it is happening because of earthquake. In addition, such loading can also happen due to lot of construction activities happening in and around of your site of interest. Same way, it can also happen during blasting, maybe because of some query or even at some construction site, some blasting is happening, that at times can also trigger increase in pore water pressure.

Now consider a situation, there were particles which were very close to each other in a soil medium, and in those particles, there was water also present in between the particles. Now, because of excess pore water pressure, which will try to actually push the particles away from each other, as a result, what will happen, these particles, which were actually in contact with each other as a result, during static condition, the ground was offering lot of resistance. So, here I am talking about the particular soil medium, particles were there very close to each other. So, you can say the medium was approximately very high relative density, such that you can ensure before setting up a foundation that the material is significantly strong, so that it can sustain overcoming load from the superstructure. Now, during earthquake loading, what will happen, because of additional loading which is coming from the propagation of seismic waves, there will be development of excess pore pressure, because of this excess pore pressure, what it will try, it will try to push actually the particles away from each other. So, initially, this was the state of the soil. Now, during liquefaction, or as a result of increase in pore water pressure, you will see the particles are pushed away from each other. So, the overcoming load remain the same, the foundation remain the same, but the bearing medium which was available beneath the foundation, which was otherwise providing lot of resistance to overcoming load. Now, in this particular case, which is defined as the state of liquefaction, all the particles, all the soil particles, have been pushed away from each other, and the gap which is there in between the particle, it is basically filled up by water, which was earlier also it was there, but now there is additional pore water pressure or excess pore pressure.

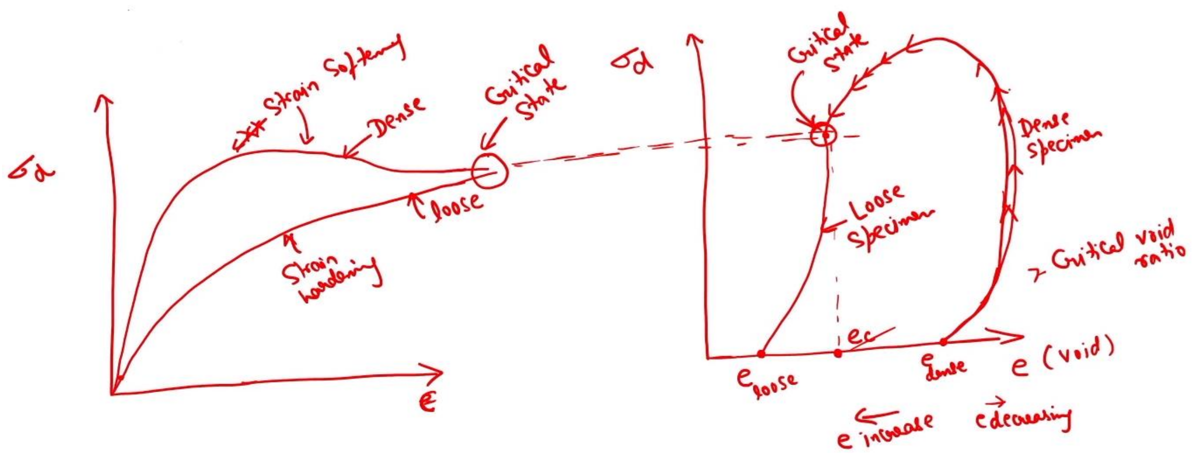
So, consider the state in which load is there, but the medium which otherwise was stable, now has been replaced by another medium, which is almost kind of slurry or liquid, particles are very far from each other, and in between the particles, there is water. So, it is in this particular state, whatever load you are applying, it will not be able to sustain that particular load. If you have a foundation, foundation will undergo either total settlement or differential settlement. If such thing is happening along the pavement, you can see differential settlement, development of cracks even at the top surface also. Bridge abutments are there, if foundation of the bridge abutment undergoes liquefaction, you can see failure of the bridge abutment, which has also been seen, lot of liquefaction related damages have been reported during 1964 Niigata earthquake as well as Alaska earthquake, and since then, lot of studies related to liquefaction occurrence and the phenomena of triggering of liquefaction at a particular site related to earthquake loading condition or even under static condition has been studied by different literatures. So, different researchers have proposed lot of understanding related to what is the triggering criteria which is helpful for initiation of liquefaction.

So, in general, this particular topic of liquefaction, in this particular course, has been divided into 4 lectures. Initial 3 lectures, we will be discussing about the state criteria, which defines what is actually the state with respect to the initial state, state whenever we are saying, it is like with respect to the loading condition, what is the state of stress in which the soil was there initially, and when the same soil is subjected to whether increase in confinement, decrease in confinement, change in deviatoric stress, how the state of stress with respect to initial state will change and subsequently how this will lead to failure, which in this particular case, we are defining as a state of liquefaction or a state when the soil from a completely stable medium has been transformed to almost like a liquid form. So, how this transition between the initial state to final state of liquefaction has been happening, we will discuss in lecture 21, 22 and 23. Subsequently, in lecture 24, we will also discuss how one can quantify the factor of safety or how one can quantify the potential of a particular site to undergo liquefaction. So, many a times, when we are dealing with micro zonation studies or development of liquefaction hazard map of a particular site or a particular region, we will be doing quantitative assessment of what is the strength soil is offering, how much is the loading condition, and based on these two comparison, we can come up with whether a particular site, depending upon its in-situ strength properties, which can be measured from a number of in-situ field investigations. So, that will help us in determining how much the liquefaction resistance of a particular site, at the same time, we will also try to find out how much is the loading, which is going to get generated during a particular earthquake. So, once the loading is there, which is going to generate stresses and in-situ strength condition, which is going to tell us how much resistance soil is going to offer. Comparing these two terms, keeping the loading criteria on same scale, we will be able to determine how much is the factor of safety, a particular soil for a given earthquake loading is going to offer. So, that we will discuss in lecture 24. We will also talk about few of the numerical about how to quantify the liquefaction potential of a particular site, taking various factors, correlations into account.

So, as far as state criteria is there, now in the earlier lecture also, we had discussed about the particle movements, change in ground, change in ground motion characteristics. In lecture 24, we will also be discussing about, depending upon the characteristics of the soil, such as relative density, liquid limit, initial moisture content, plastic limit, all those things directly will give an indication about whether a site, which is consisting of a particular soil, is prone to undergo liquefaction or not. So, this is to identify what are the particular soils which are prone to undergo liquefaction, that means if favorable conditions are given, these soils will undergo liquefaction. Now, how this will undergo liquefaction, how the initial state of the soil will change such that whatever was there under static condition and then subjected to additional loading, whether it is because of static loading or because of dynamic loading, finally reaching to its failure, that we will discuss in state criteria, which will give us a clear understanding about what are the soils based on the initial state which can be classified as liquefiable, what are the soils based on the initial state which can be classified again as non-liquefiable. So, as I mentioned, the same thing is written over here. Even if a soil meets the criteria, criteria means you can go with Chinese criteria, you can go with modified Chinese criteria, that will give you inherent characteristics of the soil based on which one can identify whether the soil, particular soil, is meeting the criteria of liquefaction or soil has the susceptibility to undergo liquefaction or not. However, whenever we go with state criteria, we are interested that, considering the initial state of the soil, considering the loading condition in a particular soil, whether the soil will be susceptible, whether the soil, what is the state in which the soil will move from the initial state in order to reach its liquefaction. At the same time, we will also be able to differentiate between the soils which are liquefiable, which are not liquefiable, or which are susceptible to liquefaction, which are not susceptible to liquefaction. So, that we will discuss in state criteria.

Now, the primary works done by Casagrande in 1936 may become the base for understanding the critical state criteria as far as the initiation of liquefaction is concerned. So, in 1936, Casagrande, basically performed strain-controlled drained tests. Strain-controlled drain tests were performed by Casagrande in 1936. So, this is the research paper one can refer to if one is interested to further go into the details, and these tests were performed on loose soil as well as dense soil. So, based on, see, primarily we are talking about cohesionless soil over here, which are also subjected to very high groundwater table. Usually considered as a worst scenario, that the groundwater table is almost reaching the ground surface. If it is deeper than that, then depending upon the depth of the groundwater table and taking into account what is the susceptibility of that particular site, what is the historical scenario witnessed during different earthquakes, one can suitably take a decision, like what should be the depth. Otherwise, maximum time, the depth of the groundwater table is considered up to the ground surface, so that it can represent the worst scenario as far as liquefaction occurrence is concerned.

So, high groundwater table, this is the favorable condition as far as the soil is concerned as well as groundwater table is concerned. Why primarily? Because the occurrence of liquefaction is more confined to excess pore water pressure. So, Casagrande, in 1936, performed strain control tests, and then, based upon strain control tests, it was observed that whenever you are doing strain control tests, so you are having deviatoric stress. Based upon the observations which were experienced during a particular test, it was observed that whenever you are doing a testing, you will have some value of deviatoric stress and some value of confining pressure also. So, accordingly, one can get an understanding about the stress-strain curve.



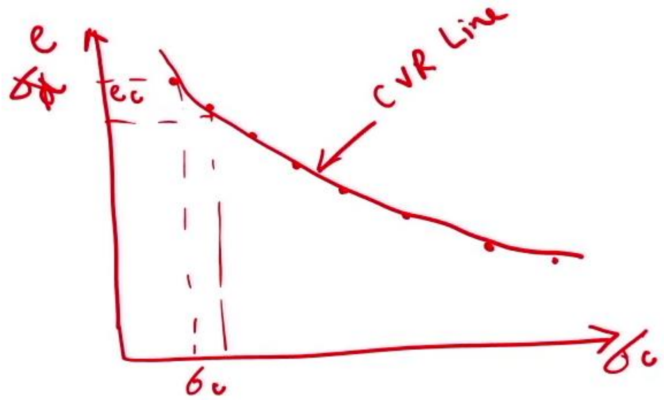
So, this is corresponding to epsilon value. Now, here if you see, so, this is deviatoric stress versus strain value. Now, when we, if we recall the behavior of cohesionless soil subjected to loading condition, primarily we will see two types of behavior. Whenever we are discussing about loose material subjected to loading, what is happening over the material will undergo strain hardening. So, you can see throughout the state of loading, this was the initial state of the soil, and then from here, we started loading the sample, subjected to increase in the deviatoric stress, which is the representation of additional loading which is coming from superstructure, surcharge load, or any other load which is primarily going to trigger additional loading in the soil medium. So, whenever soil at a particular site is loose, it will be subjected to strain hardening. So, material goes to its denser state. This is about the loose material. Now, classification of loose and dense, depending upon the relative density, one can do. At the same time, if you take dense material, so, loose material was subjected to strain hardening. You can see over here, primarily over here, that the material is undergoing strain softening. Initially, the material started taking a load. You can see over here from the initial part. Then, after reaching a peak value, you can see the material has shown a signature of strain softening. As a result, whatever loading it was taking, deviatoric stress it was taking, suddenly you will see the load-carrying capacity of the material has reduced because of strain softening, and subsequently, the material is also showing significant increase in the exterior strain values. So, this is the basic nature whenever strain control tests are done under drained condition.

Now, at the same time, if you are interested to find out what is happening at the void ratio level, because primarily, we are looking from the void ratio point of view. So, this is again deviatoric stress, and then this is again about void ratio. So, I am interested to find out what is happening at the void ratio condition. Now, here we see that whenever there is, consider this is the state of stress of the initial soil sample. Now, here we can see that the void ratio is increasing in this particular direction, e increasing. So, we are saying like as we are moving towards your left-hand side, void ratio is increasing. So, this, you can say, void ratio corresponding to dense medium, and this is void ratio corresponding to loose medium. So, here, what we saw actually, whether you are subjected dense specimen subjected to loading condition, initially, it will take the load, reach to the peak value, and followed by strain hardening, and it will continue, and certainly, there will be a stage come where it will not take any particular load, but more or less it was subjected to increase in exterior strain. Loose specimen, again, when we started loading, it was continuously taking a load, and subsequently, we will see that increase in load-carrying capacity with respect to exterior strain, that will become more or less constant. So, again, here also we will see that it is continuously almost horizontal line. So, at this particular point, you will see whether it is loose specimen or it is dense specimen. That means loose specimen is the state, is the initial state of the soil, dense specimen is the initial state of the soil. When both the samples are subjected to loading conditions, some same value of deviatoric stress, both the samples are reaching to more or less the same state. This state is called as critical state. That means, independent of whether the soil is loose or dense, both the samples are corresponding to critical state. Then, at the same time, if this is loose sample, we can see over here also where loose sample is there. So, this is corresponding to loose sample, where loose sample is subjected to loading condition. You can see there is decrease. We can see here that in this particular direction, e is decreasing, and towards the left-hand side of the screen, the void ratio is decreasing. So, whenever void ratio corresponding to loose condition, when that particular soil sample is subjected to loading condition, you can see there is a reduction in the void ratio, which can also be seen by means of strain hardening, and once it reaches more or less to the critical state, it is more or less, there is no further decrease in the void ratio.

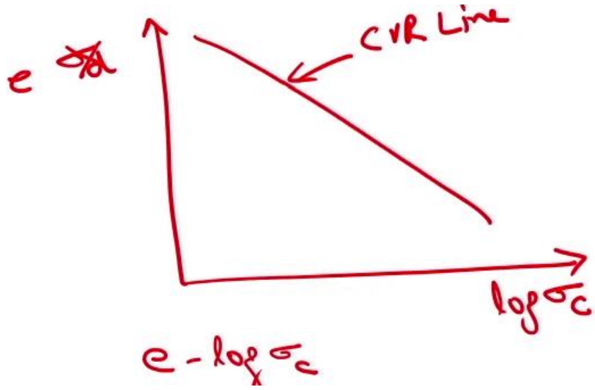
On the contrary, there is dense specimen also. So, this specimen initially started taking loading, which is indicated by decrease in void ratio subsequently. So, you can see over here, this particular sample, even if we take that the two lines are almost reaching to the same state. So, we can see over here. So, this is corresponding to dense specimen, and this one is corresponding to loose specimen. Now, here both the samples are reaching to the same site condition, same critical state. So, this particular void ratio, whether it is a loose specimen after strain hardening is reaching, or a dense specimen after strain softening is reaching, or after dilation and contractive behavior is reaching, this particular void ratio, it is called as e critical or e_c. So, I am calling this as critical void ratio, and again, this particular state is called as critical state. So, independent of the initial state of the soil, depending upon the initial state of the soil, it will define whether the sample will undergo strain hardening or strain softening, but finally, all the samples will be reaching the same state of critical state. All the samples are reaching and corresponding to this particular state, this e_c is called as critical void ratio.

Now, if we try to understand what is happening over here, we took a sample, confined the sample to a particular confining pressure, and when the sample is confined, and the confinement is applied to the particular sample, then we started subjecting it to deviatoric stress increase and measured how much the exterior strain changed. Corresponding to these things, the loose specimen will show the signature of strain hardening or contractive behavior. After reaching a particular state of stress, there will not be any further increase in the deviatoric stress or load-carrying capacity. Dense specimen, on the other hand, initially started taking a load, reached a peak value, after that there is dilation, and then subsequently, even though the sample ratio is significantly high, there is no further increase in the deviatoric stress or no further subsequent decrease in the deviatoric stress. Same if you see with respect to the deviatoric stress versus void ratio line. So, we can see two specimens: when a loose specimen is there, it was subjected to strain hardening, and there is a reduction in the void ratio, reaching to its critical state. On the other hand, dense specimens are there, which, initially, will take the load. So, you can see, this particular side, it is basically an indication of strain hardening, these two peak values, and then subsequently, this is the strain softening part, reaching to the critical state. Now, as I mentioned, this is corresponding to one value of confining pressure. That means, for each value of confining pressure, sigma c, I can write. So, in the end, you are getting, corresponding to one value of sigma c, one e suffix c.

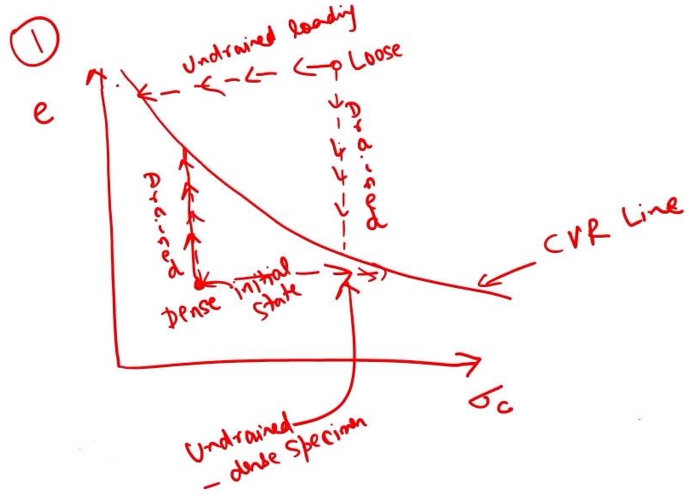
That means, corresponding to one value of confining pressure, for a sample, whether it is a dense sample or loose sample of the same soil. Remember, the soil remains the same; depending upon the relative density of the soil, we are categorizing it as a loose specimen or dense specimen. So, the sample remains the same. Depending upon the initial state of the soil, before you started applying the load, there will be a change in the path which the sample takes before reaching its critical state. And once it reaches the critical state, independent of whether the initial state was loose or dry, all the samples will be reaching the same void ratio that is called as critical void ratio. Now, repeating the same procedure. So, this is corresponding to one confining pressure. Repeating the same procedure, the above procedure, for different confining pressures, that means, you have done at one confining pressure, obtained the value of sigma c, critical void ratio, again repeat the same set of experiments corresponding to different values of confining pressure, you will get another value of critical void ratio. Again, repeat the same procedure for different values of confining pressure, every time you will get different values of void ratio, that means, critical void ratio. So, collectively, based on this, what one can obtain is a plot of, and remember, all these void ratios are defining a state of soil which is called as critical state. So, independent of whether the soil was loose or dense, all the soil samples ultimately will reach the critical state. So, e_c is the critical state void ratio.



Now, this is the value of void ratio, deviatoric stress, and then corresponding to sigma c, what we will get over here is the critical void ratio line. So, corresponding to deviatoric stress and the value of confining pressure, we will get some value of void ratio. So, this is like the critical void ratio joining all the critical void ratios. So, we will get something like this. So, this is called as the critical void ratio line. I am calling it a line because, generally, we try to plot this in terms of deviatoric stress versus confining pressure in terms of the log value.



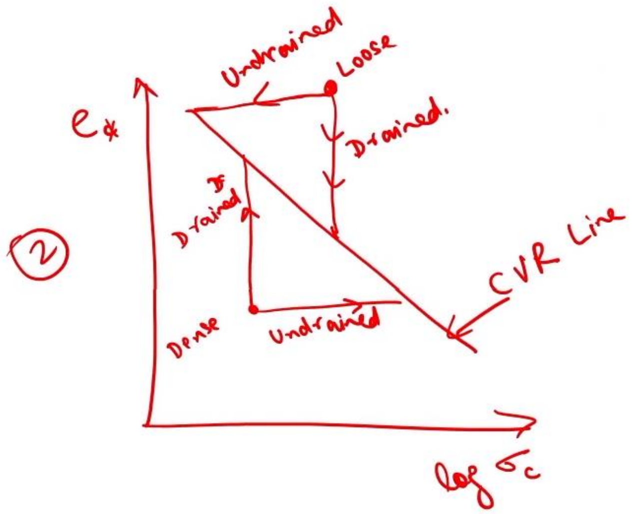
So, this is log sigma c, and then this is the value of sigma d, you will get this particular line. This is actually void ratio; here also, it is void ratio. So, void ratio, so, you can see over here, corresponding to this value of confining pressure, sigma c, this was the value of critical void ratio. Corresponding to this confining pressure, this was the value. So, joining all the locus, this line is basically a representation of the locus of critical void ratios for different confining pressures. Same thing when you are developing at log sigma 3 or the logarithmic confining pressure e log sigma c curve. So, this is called as the e log sigma c curve. This is called as, this is then appearing as a form of a line, which is called the critical void ratio line or CVR line. So, this is also called as CVR. Now, with respect to this particular line, now we have understood that, based on the critical state of the soil, let me draw it once again.



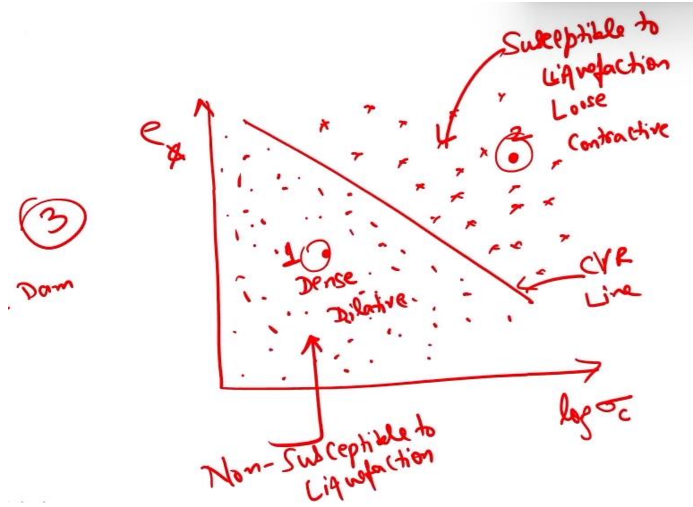
Based on the initial state of the soil, the sample was subjected to strain hardening or strain softening, finally reaching the same state which is defined by a critical state having a certain value of deviatoric stress, a certain value of confining pressure, and a certain value of void ratio. Joining the points, explaining or representing different values of confining pressure, the critical void ratio at different confining pressures, we try determining the CVR line. Again, I am calling it the CVR line. As I mentioned, when you are developing it as the E sigma c curve, it will not appear as a line. Once you take log sigma c, it will appear as a line. Now, with respect to this particular line, suppose for a particular soil, we have obtained this particular line. As I mentioned, this is basically a representation of a number of samples of the same soil, with changes in the initial condition or initial state of the soil, some representing a loose state, some representing a dense state. The same soil at different initial states subjected to loading, deviatoric stress, and before that, it was corresponding to the same value of confining pressure you have applied, and after that, you started increasing the deviatoric stress. Every time we get critical void ratio, joining these points. So, this is basically going to suggest whatever is the sample, whatever is the initial state of the soil, finally, the soil sample will reach this particular part, that is the critical void ratio line, because every sample is destined to reach its critical void ratio. Now, once the critical void ratio line is known to us, take two samples: one is a dense sample of the same soil corresponding to which this critical void ratio line has been established. So, it is like, once for a particular soil, the critical void ratio line is established, now we are interested to find out that with respect to the initial state of the soil, what path the sample will take to reach its critical state. Now, this is one sample which is called a loose sample, and at the same time, we can have, so this was a loose sample, and we can have a dense sample, and we can additionally have two samples. Primarily, we can have two types of samples: either the sample can be loose or the sample can be dense, and the critical void ratio line of a soil is developed already based on a lot of experimental works. So, we can see based on this now, here we see two types of loading, actually. So, at a particular side, the soil is either loose or the soil is dense. At the same time, depending upon the boundary condition, depending upon the drainage path available, that means, your soil is subjected to loading, deviatoric stress, and now because of this additional loading, whether the soil sample is allowed to drain excess pore water or dissipation of excess pore pressure is allowed, or it is not allowed, accordingly, we will have two types of conditions: one is a drained condition and the next one is an undrained condition. So, when we talk, when we think from the drained condition, the drained condition means we are allowing the water to undergo drainage, as a result of which there will be a change in the void ratio.

So, if this is a sample, now this particular sample is subjected to a drained condition. Definitely, when it is subjected to a drained condition, there will be a reduction in the void ratio, but considering this is a dense sample which will be subjected to strain softening, there will be an increase in the void ratio. That is why we can see there is a change in the void ratio, or rather, an increase in the void ratio based on the initial state for the drained condition. Remember, this is the drained condition. So, whatever external load you are applying, it is basically resulting in bringing the particles together close to each other. As a result, there will be a reduction in the void ratio, or in the case of loose dense samples, there will be an increase in the volume. So, there will be an increase in the void ratio. Subsequently, the sample, when subjected to the drained condition, is reaching, taking this particular path, and it is reaching its critical state. Now, if the sample remains the same, but suppose the drainage path is not there, you are having only pervious or impervious layers, what will happen is the sample will be subjected to the undrained condition. So, this is the path which the sample will take under undrained loading for a dense specimen. The sample is still dense, but now drainage is not allowed. So, once drainage is not allowed, certainly, we cannot see any change in the void ratio. The void ratio is not changing. Now, considering confining pressure, that will increase because the dense specimen is there. So, it will be subjected to the development of negative pore pressure and then, subsequently, there will be an increase in the value of confinement. This is about strain softening.

Now, over here, we will see another specimen that is a representation of a loose specimen. So, again, for the loose specimen, whenever it is subjected to the drained condition (remember, this is the drained condition), the loose specimen means the sample, once it is subjected to loading, will undergo strain hardening. In strain hardening, the sample will show contractive behavior, and there will be a reduction in the void ratio, which is indicated by a downward arrow for the loose specimen. Subsequently, the sample will reach its initial state, the critical state. On the contrary, the loose specimen, when subjected to undrained loading condition (undrained loading), again, the sample is reaching its critical void ratio, but rather than changing in the void ratio (because this is an undrained condition), water cannot go anywhere, and there cannot be a change in the void ratio. However, in this particular case, because it is strain hardening, there will be the development of pore water pressure, and subsequently, a reduction in the effective stress, or subsequently, a reduction in the confinement. So, confinement will reduce in the case of a dense specimen, while confinement will increase because of dilation. As a result, you are seeing that the initial state of the soil is represented by right-side movement for the dense specimen until it reaches its critical state. For loose specimen, because of the undrained condition, there will be the development of pore water pressure, and as a result, your confinement will reduce. Subsequently, the sample's state will be represented by left-side movement of the sample until it reaches its critical state.



Now, if you are looking from the e-log σc curve, so now this is the critical void ratio line. Again, I can tell over here, this is your dense specimen. It will go like this for the undrained condition and like this for the drained condition. This is for the dense specimen. For the loose specimen, you can see over here. So, this is again for the undrained condition, reduction in the effective stress or change in the reduction in the void ratio. So, this is corresponding to the drained condition, and this is corresponding to the undrained condition. So, now, collectively, what we have understood is that whether the soil is subjected to drained condition or undrained condition, whether the initial state of the soil represents its loose state or dense state, every soil sample will reach its critical void ratio line. Remember, this is the critical void ratio line. Now, if you are looking from a liquefaction point of view, primarily, the phenomenon of liquefaction is reported in loose soil. So, taking that into account, if I am interested in finding out, based on the initial state of the soil, what I can get, there are samples.



This is your e-log σc curve. This is my critical void ratio line, and the sample is there. Now, I can decide whether the sample is over here or the sample is over here. I am calling this sample number 1 and sample number 2. Based on our understanding so far, we can say sample 1 is a representation of a soil corresponding to the initial state of the dense specimen. On the contrary, now we need not discuss here which particular stress path the soil will take to reach its critical state. We have seen in the first and second figures, this is figure 1, this is figure 2, and this is figure 3. So, we have clearly seen that whether the soil sample is loose or dense, whether it is subjected to drained condition or undrained condition, all the samples will reach the critical state. Now, depending on the initial state of the soil, some samples will reach the critical state, maybe at higher void ratio, corresponding to the other. That also depends on what the level of confining stress you applied to a particular sample before you started applying the deviatoric stress. Now, coming over to figure 3, we have established the critical void ratio line for a particular sample, and then, at times, I am interested in finding out which particular soil sample, out of these 1 and 2, is actually susceptible to liquefaction. Susceptible to liquefaction means if failure loading conditions are given to the sample, which of the two samples are subjected to or can undergo liquefaction during the state when it is reaching its critical void ratio line?

See, the critical void ratio line, finally, every sample will reach, but something is happening between the critical void ratio line and the initial state of the soil, where the phenomenon of liquefaction will also come. We will discuss that later. Now, here, two samples are there. This is a representation of a loose sample, and this is a representation of a dense sample. This is based on the initial state and also considering the fact that dense specimens are less prone or not prone to liquefaction. This particular zone, which is located, or all the samples which are located below, which are represented by the critical void ratio line, and corresponding to, so you can see e_c or void ratio. Now, I am not writing here e_c; initially, I wrote it because whenever I am taking any sample (1, 2, or in this particular case, also dense or loose specimens), I am also trying to see how the state of the soil changes. So, not every time when it is subjected to drained condition in loose soil or in dense soil, not every time when the state of the soil or the state of stresses changes will it be represented by the critical void ratio. So, the critical void ratio is just one value, but at the time when you are loading this particular soil sample, the void ratio will change, which will not be called the critical void ratio. And since this change of stresses, we are representing by upward and downward lines for dense and loose specimens, respectively, I am not marking this as the critical void ratio.

So, this particular soil sample or all the soils, whose initial state represents that the sample should be located below the critical void ratio line, are representations of non-susceptible, non-susceptible to liquefaction. So, any sample based on the initial state of the soil, once it is falling below the critical void ratio line, the sample is declared as not susceptible to liquefaction. On the other hand, all the samples which are located above the critical void ratio line, as mentioned over here, these are representations of corresponding to any value of confining pressure. Thus, the void ratio can be higher or lower. All these are susceptible to liquefaction. So, you can call it as susceptible to liquefaction. So, based on the initial state and demarcating the boundaries between loose and dense specimens, one is able to identify whether the soil is susceptible to liquefaction, or it is not susceptible to liquefaction. Now, whether the soil will undergo liquefaction or not, that we will discuss. As I mentioned earlier, we will discuss in lecture number 24. How to quantify the liquefaction potential of a particular site and determine its factor of safety. That will be discussed in lecture 24.

Now, based on this, though we can identify whether the soil will undergo liquefaction or not, here we can say this is dense or dilated behaviour; this is a loose specimen or contractive behaviour. Now, based on this, one can write that the critical void ratio line demarcates the boundary between a loose sample and a dense sample, or it can demarcate the boundary between samples which are susceptible to liquefaction and which are not susceptible to liquefaction. Now, one limitation which was later highlighted corresponding to the critical void ratio line is that though it defines the state of the soil, whether it is loose soil or dense soil, to reach the critical state, later on many of the actual field studies suggested that there were samples which actually were plotted based on the initial state below the critical void ratio line. Whenever these samples were actually subjected to actual earthquake loading conditions, they had actually undergone liquefaction. That means, at many instances, one instance was during the Fort Deck dam. There was an earthquake during which even the samples that were located below the critical void ratio lines were found to have undergone liquefaction. It is primarily because the critical void ratio line is basically a representation of the critical state of the soil and with respect to the initial state of the soil.

So, as I mentioned some time back, when you start with the initial state of the soil, whether the sample is 1 or the sample is 2, all the samples, how these are reaching the critical state, and during this particular stage of loading, whether any particular stage comes where the resistance offered by the soil and the state of stress or the loading by external earthquake loading, what is the relative comparison, that primarily was not clearly identified corresponding to samples during the Fort Deck dam. So, for Fort Deck dam , some samples which were identified as non-liquefiable as per the critical void ratio line were later found to be liquefiable. So, as a result, Case and 1930s, later on, work they identified that this is the inability of strain control drain tests to capture all the phenomena which are happening for loose samples as well as dense samples, which are actually responsible to understand the initiation of liquefaction. So, one is interested to find out how the initiation of liquefaction at a particular site has happened or is going to happen during a particular earthquake. Then the strain control drain test, so far, we were discussing about when this understanding of the critical void ratio line was proposed, perhaps could not capture all the phenomena which were actually covered in the strain control test. So, later on, Castro, which we will discuss in lecture number 22, in 1960 came up with many more tests and tried to find out that similar to the critical state of the soil, there is an additional boundary condition based on which we can understand the initial state of the soil and how the initial state of the soil changes with respect to loading conditions, whether it is corresponding to, I mean no more it will be called as susceptible or not susceptible. Rather, it is basically corresponding to how much is the confinement available and how much is the initial state of the soil which is available to offer resistance. So, we will discuss further detail in lecture 22 about the steady-state line or steady state of the soil, steady-state line. So, with this, I will conclude this particular lecture.

So, in today's lecture, we have discussed that primarily when we are discussing the occurrence of liquefaction, there are two types of conditions the soil has to meet. One is that the soil should be cohesionless, and secondly, the soil should have a very high groundwater table because, after all, it is more related to the development of excess pore pressure. Now, in actual site conditions, where the soil may or may not undergo liquefaction, the soil sample can be in a loose state or the soil sample can be in a dense state. Whenever these samples are subjected to strain control drain tests, we identify that both the samples are reaching the critical state. The dense sample is subjected to initial load, followed by peak value, and then subjected to strain softening. The loose sample, on the other hand, will be subjected to strain hardening and subsequently reaching the same state, more or less, where the dense sample has reached at very high axial strain. The same way, if we look from the void ratio point of view, initially, there will be a reduction in the void ratio in the dense specimen reaching the peak value and then subsequently an increase in the void ratio. The loose sample, since the beginning, as the loading progresses, there will be a reduction in the void ratio. That means whether the sample is loose or dense, for a particular soil, at one value of confining pressure, you will have one value of the critical void ratio. Repeat the same set for the same soil but at different confining pressures, and you will get a complete locus of n number of points, which are representations of the critical void ratio at one value of confining pressure, and subsequently, so n number of critical void ratios corresponding to n number of confining pressures.

So, by joining all those points, we will get one sigma c, that is, confining pressure plot, a curve which on the e-log sigma c plot is also represented by a line defined as the critical void ratio line. So, this critical void ratio line is the locus of all points representing the critical void ratio or critical state of a soil sample, where whenever it is subjected to different levels of confining pressure. Now, when the same sample at a particular site, whether the soil sample can be loose or the soil sample can be dense, it can also be subjected to drained or undrained conditions. When we are talking about the drained condition for the loose sample, then it will be subjected to a reduction in the void ratio, subsequently reaching its critical void ratio line. Whenever it is subjected to an undrained condition, there will be a buildup of pore water pressure, subsequently reducing the confinement. On the contrary, when the dense sample is there, subjected to strain softening, there will be a reduction in the confining pressure. Subsequently, the sample will reach its critical state. Whenever the dense sample is there and also subjected to the drained condition, then there will be, because of strain hardening softening, an increase in the void ratio as the sample reaches its critical void ratio line.

In the end, because in some of the instances the identification of whether the soil is susceptible or not, based on the critical void ratio line, failed, then later on it was firstly identified that it was primarily because of strain control tests, which were available at that particular time. So, they could not capture properly the variation with respect to the initial state of the soil to the critical void ratio line or the critical state. So, later on, in 1960, Castro performed a lot of other tests and came up with another understanding related to the steady-state line, which will help in identifying, first, how the initial state of the soil changes and how the initiation of liquefaction can be studied with respect to the initial state of the soil and with respect to additional loading, whether it can be because of static loading or dynamic loading.

So, that we will continue in lecture number 23 and lecture number 22. So, thank you everyone. We will stop here. Thank you.