Applied Seismology for Engineers

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Week – 09 Lecture - 04

Lecture – 24

Hello everyone, myself Dr. Abhishek Kumar. Welcome to lecture 24 of the course Applied Seismology for Engineers. In today's course, we will be discussing about one important induced effect of earthquake, primarily related to soil, which is called as liquefaction. Generally, whenever waves generated at the source reach to a particular site, these waves will undergo slight modifications and then finally reach to the surface. Now, depending upon whatever medium is available onto the surface, whether it is a soil medium, whether it is foundation, whether it is superstructure, each of these mediums will interact with the wave or the disturbances which are created by the wave in the propagation medium. As a result of which, there will be loading which will be created in the medium because of the propagation of various waves. How the particular medium, whether it is soil, whether it is foundation, whether it is pile foundation, or whether it is superstructure, how these are going to respond will decide how the system will remain in its position. It may undergo partial damage; it may undergo collapse. Similarly, in terms of buildings as well, if seismic loading is quite intense in terms such that the building is undergoing too much of shaking, subsequently there are lot of displacements in the building, cracks are there in the building, the building will undergo complete collapse as well. So, it is basically how a particular building or a structure or a soil medium is going to respond due to earthquake loading condition that will define the process how the earthquake loading is actually contributing to the existing structure, existing soil.

As we discussed in seismic hazard analysis that when waves reach to a particular site, there are different ways in which the interaction of the wave with the medium will come into light. Primary example is excessive ground shaking. The seismic waves when being modified because of local soil and reaching onto the ground surface, these seismic waves will cause too much of ground shaking even in the foundation, the superstructure. So, that is one kind of induced effect. It is actually not the effect because of the seismic wave but the resultant of how the system available at a particular site is going to respond to the seismic waves. Similarly, you can have landslides. Again, there is a medium, there is a structure which is going to respond to the seismic waves which are actually being applied to that particular medium. We will discuss how different kinds of landslides may happen, but each of these landslides are basically the response of your soil, response of your structure, response of your rock which are actually the part of your structure corresponding to the earthquake loading condition. If the medium is quite intact, it is having enough strength; certainly, the medium will not undergo any kind of failure. You can say there was no landslide, the material was safe against landslide occurrence.

Similarly, if the ground shaking at the surface is lower, you can say the induced effect due to ground shaking will be significantly low. In similar manner, another induced effect is called as liquefaction. As the name suggests, it is the phenomena in which by virtue of earthquake loading which will be applied to the soil medium and bring into account because there are soil medium in this pore species which are available in the soil medium, there will be water present in the medium. So, what happens when earthquake loading is applied to the soil medium which is also having water in the pores. In general, during static condition, when you applied a load, the pore water, it will come out through the pores, reach to the drainage path, and then it will dissipate because in static loading condition, the load will also take time to apply it, and then subsequently, the pore water will also get time to dissipate. However, in case of earthquake loading condition, the scenario significantly changes. That means you are applying some loading, we have already discussed what is the rate at which, what is the velocity at which the waves are passing through a particular medium, that clearly gives an indication what is the duration for which the earthquake loading will be applied in a particular medium, it will be some fraction of seconds.

So now, consider again a saturated medium, primarily in sandy medium because in sandy medium only generally during static condition the time is significantly larger, the permeability of the medium is also facilitating dissipation of pore pressure with respect to loading. However, in the same sandy medium, whenever there is earthquake loading or waves which are passing through a particular medium, if you recall the propagation of wave through a particular medium will cause particle oscillation. So now, there is wave which is passing through a particular medium causing some kind of disturbance in the particle which will actually induce pore water pressure, keeping that the duration of such loading, the duration for which the seismic wave is actually inducing displacements in the particle through which it is passing, it is of the range of some fraction of seconds. That fraction of second may not be enough for pore water pressure to dissipate. So, what will happen, the pore water pressure, which was the pore water which was actually building a pressure inside the soil medium, that will actually try to push because there is no drainage path, there is not enough time for this pore water to reach to the drainage path and get dissipated within that small duration of time. So, what will happen, this pore water which is now under tremendous pressure because of seismic loading condition, will start pushing the particles. As a result or the end effect of seismic loading on soil medium, primarily cohesionless soil medium, we will see that during static condition, a ground which was significantly labelled, it was providing enough strength to superstructure or any other kind of loading whether it was dead load, live load, it was offering significant resistance because even though there was building up of pore water pressure, significant time was there for that pore pressure to get dissipated.

However, in case of earthquake loading, the dissipation cannot happen because the rate of loading is so fast that even considering very high permeability, the time which requires participation is not available in the medium. As a result, as I mentioned this built up of pore water which is building up within the pore, pores actually will try to push soil particles. So initially, before earthquake loading came into picture, thus the medium was quite stable, it was offering resistance whether you can say it was supporting a building or it was supporting maybe wind turbine or it may be supporting a car parked on the particular soil medium. So, all these structures or all these kinds of loading were more or less in stable condition, suddenly an earthquake loading is applied to that particular soil, if there is a building, consider there is a building and the foundation of that building experiences building up of pore water pressure. As a result of this excess pore pressure, the particles have now been pushed away from each other, so initially the particles were in relatively confined position, now have actually moved to quite loose state. So, if you look at that particular soil, it will almost look like a liquid. So, liquid is basically representing a phenomena in which the soil, which was otherwise during static condition, was providing sufficient strength, the medium itself or the superstructure itself was significantly in stable condition, but because now the soil has liquefied or the soil has undergone completely into state of liquid, any kind of superstructure which is actually located. So, consider there was a building over here and corresponding to this particular building, there was some foundations which were actually gaining resistance from the soil, whether you say in terms of settlement, you say in terms of bearing capacity, but sufficient resistance was there from the soil medium. Now, consider this was during during static condition, same soil remains same but now you have applied additional loading because of seismic loading, you have applied, as a result of this particular seismic loading, there was building up of pore water pressure, when this pore water pressure built up, there was development of excess pore pressure within this particular medium.

So, soil particles which were very close now will start moving away from each other. This particular medium, same medium during dynamic loading, during static condition has actually turned the medium into liquid state. Liquid state means soil having very high inter-particular space and in between those particles which are separated by significant distance is only water is there. So that is almost like a liquid, or you can say like it is almost kind of slurry which has capability of flowing. Now, initially in static condition the material was offering significant strength. Suddenly you can consider this material which was otherwise very stiff, has been replaced by another material which is almost like a liquid. In such a case, what will happen to the foundation? What will happen to the building? Because the foundation medium is not able to offer resistance, which otherwise it was offering, suddenly you will see if it is happening all throughout, you may see uniform settlement. If it is happening on one side rather in comparison to the other side, you may see the building has undergone tilting, or there is a sign of uneven settlement. If it is happening throughout the building uniformly, you may see too much of total settlement; the building itself has sunk into the ground, maybe half a story, one story, maybe less than that, maybe more than that. So that means, not because of the strength of the building, but because of loss of strength in the subsurface medium, the superstructure has undergone failure.

Now, the same thing we can also understand. If there is a wind turbine above it, that will also undergo differential settlement or too much of total settlement. If there is a parking space, you parked a vehicle in the parking area; suddenly there was an earthquake loading. As a result of this earthquake loading, the soil which was available in the parking has actually turned into liquid. You parked your vehicle over that; what will happen to that particular car? That will simply sink into the ground. So many a time, if you search, many a time you will see that particular parking area turned into liquid, as a result of which whatever vehicles were parked, those were actually sunk into the ground. Many a time, you will see, again because of excessive building of pore water pressure, there will be loss of material, or the material turned into liquid at a certain depth below the pavement. Suddenly, you will see after a particular earthquake loading, you will see some significant portion of the pavement has shown wider cracks or has actually sunk into the ground. So again, ground subsidence is another way in which induced effects of earthquakes can be visualized at the site.

So liquefaction, as I mentioned, not only relates to open areas but many a time because of buildings, because of parking spaces, or because of other utilities which are actually supported by the ground. In this particular case, the ground itself is losing strength, and subsequently, there will not be significant support from the ground, leading to failure. That is why it is important to understand the liquefaction potential of a particular site. When we say liquefaction potential, that means we are interested to know how much potential the site has to undergo liquefaction. If we are able to find out the liquefaction potential of a particular site and based on our assessment, we found that the site is sufficiently safe against liquefaction, that means you need not worry about future earthquake loading during which the chances that the soil may undergo liquefaction are very less. On the contrary, there are sites where even if you find that the sites have a very high potential to undergo liquefaction, you need not wait for actual earthquake loading to be induced at a particular site and undergo failure in terms of liquefaction, ground subsidence, or total or differential settlement.

What we will do, if we have found out that the site is potential to undergo liquefaction, we will take that particular site and treat that particular ground. Whether you can go with ground improvement techniques or there are other ways in which you can actually enhance the in-situ strength of the soil such that when the actual earthquake loading is applied onto the soil medium, the soil medium will have sufficient strength in comparison to the stresses which will actually be mobilized in the soil during earthquake loading conditions. So that will ensure that even though the soil initially was very soft, by means of treating the particular soil, we have actually enhanced the in-situ strength of the soil. Once the in-situ strength of the soil has been enhanced, you can actually go for the construction of a superstructure, ensuring that during a particular earthquake loading, this particular soil medium will not undergo liquefaction. So there will not be any kind of failure which otherwise would have happened on that particular soil medium because of the liquefaction phenomenon.

So in today's class, we will be discussing primarily how to quantify the liquefaction potential of a particular site. Further, there are different methods in which the liquefaction potential of a particular site can be found. As far as this particular lecture and this particular course are concerned, we will be focusing more on a stress-based approach, like how one can find out the liquefaction potential of a particular site.

So let us look further into the topic. Liquefaction, as I mentioned, as per Sladen et al. 1985, is defined as a phenomenon wherein a mass of soil loses a significant portion. Please note that the mass of the soil, which was actually offering resistance to overcoming load, but during earthquake loading condition, has actually lost a significant portion of its shear strength. Now, when shear strength is not there, how is the soil going to face the shear stresses, which are otherwise going to be mobilized in the soil medium due to overcoming load? So it is basically the phenomenon in which the soil medium or the mass of the soil, which we are targeting, actually loses a significant portion of its shear strength when subjected to monotonic loading, cyclic loading, or shocks. So, it need not be only earthquake loading conditions. Many a time, construction activities generating significant vibrations in terms of amplitude as well as frequency content may also cause building up of pore water pressure similar to liquefaction. During monotonic loading also, many a time, if you are driving a pile, again, this kind of situation where building up of excess pore pressure may come. So, it is more or less related to monotonic or cyclic loading conditions. Cyclic loading, as I mentioned, during earthquake loading conditions, will be cyclic, or the nature of the loading will be cyclic. As we discussed in ground motion characteristics of vibration, we can also find out what is the frequency content of harmonic motion, what is the duration of motion, bracketed duration, or significant duration; all those things we can find out.

So, such a motion which is actually showing cyclic nature, whether it is related to earthquakes or any other nature, many a time, because of blasting which is happening for different purposes, those can also cause a state of liquefaction in the soil medium. But certainly, that will depend on how much loading has actually been applied to the soil medium. That loading and in-situ properties of the soil medium collectively will decide whether the soil will undergo liquefaction or it will not undergo liquefaction. So, the soil has lost a significant portion of shear strength, and after losing that strength, it is almost flowing in a manner resembling a liquid. Now this process, that the soil has turned from stable to almost liquid form, like initially it was in a solid state, but because of building up of pore water pressure, pushing the particles away from each other, now it has turned into a liquid state. How long will this liquid state continue? Till the time shear stress is acting on the material because of external loading conditions or subsequently the built-up pore water pressure in the soil medium, or in the soil pores, is getting dissipated. Because the phenomenon of liquefaction started as a result of building up of pore water pressure.

So as far as the excess pore pressure which actually has built up in the soil medium, unless that gets dissipated, that gets over, whatever excess pore pressure was generated will reach some drainage path and dissipate. Once it is dissipated, then you will say the phenomenon of liquefaction is complete. Now, there is no more continuation of the phenomenon of liquefaction. So, it is not like once the liquefaction has happened, the soil will be in a continuous state of flow. That will not happen. So, the state of flow will only continue till the moment the dissipation of pore pressure is not complete. Once the dissipation of pore pressure is complete, there will not be further movement, whether you say in terms of large displacement or in terms of flow of material. That will completely arrest at the moment the dissipation of pore pressure is complete. So, acting on the soil medium, such that the external loading condition or the loading because of the earthquake has actually been removed. So, in such a case, what will happen? The shear stress acting on the material is also reduced as low as the shear strength of the soil, which is available in its in-situ condition. So initially, you applied some loading condition as a result of which there was a building up of pore water pressure pushing the soil particles. The pore water excess pore pressure built up will start dissipating. At the same time, there will be a reduction in the earthquake loading condition. So, dissipation of pore water pressure and reduction in the earthquake loading condition subsequently will end the phenomena of liquefaction at a particular site, and then the soil will be left with some in-situ strength of liquefied soil.

Now, there are two ways in which liquefaction generally happens at a particular site, if you say in terms of the triggering mechanism. One is flow liquefaction, as the definition suggests. Here, the flow liquefaction occurs when the shear stress required for static equilibrium. So, how much shear stress is required for static equilibrium is greater than the in-situ shear strength of the soil in its liquefied state. The soil has undergone liquefaction. Whatever shear stress you are applying from the external loading condition is actually more than the in-situ shear strength of the soil. As a result, what will happen? The soil keeps on undergoing failure, resulting in large deformation. So, such kinds of failures, where the in-situ shear strength is less than the shear stress applied because of the external loading condition, you call it flow liquefaction.

Another phenomenon because of, again, liquefaction is cyclic mobility. So, it is generally related to confinement. So, where you see it occurs when the static shear stress is less than the shear strength of the soil in a liquefied state. So, even though the shear stress is less than the strength of the soil in a liquefied state, but because of the loss of confinement, again, you will see there is the development of cyclic mobility in the medium. Again, this cyclic mobility will also result in large permanent deformations. So, these are actually the signatures one sees at a site that has undergone liquefaction. In order to see what characteristics the liquefaction phenomenon has triggered at a particular site, and then subsequently there are ways in which one can even say whether it was cyclic mobility or flow liquefaction, in which the mechanism of liquefaction is triggered at a particular site, can be understood. So, cyclic mobility, it can be triggered both by cyclic as well as static loading conditions.

Now, there are different factors that actually govern the liquefaction potential of a particular site. These include earthquake loading conditions, as we know that the phenomena of liquefaction or the application of external loading, which is actually mobilizing shear stress in the soil medium, is solely because of earthquake loading conditions. Higher is the earthquake loading condition, you can say, development of higher shear stresses will be happening in the soil medium. Same with respect to the degradation in the material properties. So, even if there is a reduction, the amplitude of loading is significantly low, but the duration is more, then again also you can see there is degradation in the material properties, as we see in the dynamic soil properties. So again, that will also control whether the soil will undergo liquefaction or not. The third one is the groundwater table. As I mentioned, the entire process of liquefaction is happening because of the building up of pore water pressure. So, if the groundwater table is at a significant depth, there will not be a development of pore water pressure, or excess pore pressure will not be there. Then that will not trigger the liquefaction phenomenon.

Soil type, again, if the soil is generally cohesionless soil, as mentioned about potentially liquifiable, if you talk about plastic low to medium plastic soils are also there. Those basically offer resistance to liquefaction. So, depending upon which type of soil is available at a particular site can also give an indication about whether, if the suitable earthquake loading conditions are generated at a particular site, the soil will undergo liquefaction or not, primarily because of its type.

Then relative density. So certainly, media that have very low relative density or soft type of medium, that means the medium's initial resistance was already significantly low, over that you have applied excess pore pressure development. Then certainly those kinds of material will undergo liquefaction much earlier or much easier than the soil where relative density was significantly high.

Particle size distribution. So, usually, if a large range of particles is present in the medium, then bigger pores will be occupied by smaller particles. So, there will be some particles which are actually available in the pore species in comparison to the soil medium which has the same sort of soil particles. So, the chances of having voids will be more in comparison to the soil where the same size of soil particles are present. So, you can say that the soils where the same size of particles are present will be more prone to liquefaction than where different-sized particles are present in the soil medium.

Placement condition, deposition environment also decide where the soil is present. What are the surrounding conditions? That will also define whether the soil has the potential to undergo liquefaction or not.

Drainage condition. If drainage condition is sufficiently enough, then there will be ease to undergo dissipation of pore water pressure. If that is not there, then the development of excess pore pressure will dominate.

Same with respect to confining pressure. If confining pressure is there, that will also provide resistance to liquefaction because as you are going deeper and deeper, with an increase in confinement, you will see the liquefaction potential of the same soil, even though the properties of the soil medium remain the same. But because of confinement, there will be a reduction in the potential to undergo liquefaction.

Particle shape. Generally, rounded particles will not have such good contact with respect to adjacent particles, so they can be separated very easily in comparison to elongated particles or where the particles can get interlocked with respect to the neighbouring particles.

Aging and cementation. Generally, when you leave a particle because of the precipitation of different salts in the medium, there will be some kind of cementation happening between the particles. As a result, it is not only the particle but also particle-particle contact. What is the nature of this particular cementation which generally comes with aging? So, you will see some particles which are freshly deposited and some particles which have been there for quite some time. So generally, older particles will, because of cementation and aging effect, be able to offer more resistance to the building up of pore water pressure and its dissipation in comparison to younger deposits.

Historical environment. Again, it has been seen that some sites that have already undergone liquefaction and, after liquefaction, have now deposited a relatively denser medium. Then, for such soil to again undergo liquefaction under the same loading condition will be relatively less because now the medium has undergone a relatively denser state.

Building load. Again, what kind of loading condition is also coming onto this from the superstructure, what surcharge you have applied, will also define what is the potential of that particular site to undergo liquefaction.

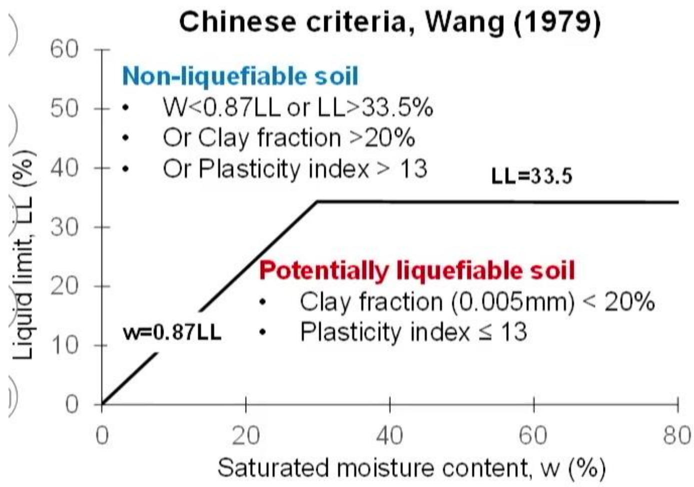
So, what are the consequences? Why is one interested to know whether the site has the potential to undergo liquefaction? Because when liquefaction occurs at a particular site, there are consequences which can be witnessed easily. Some of the consequences which can be witnessed during earthquake loading conditions, primarily when liquefaction has occurred, include settlement. As I mentioned, settlement is primarily important because the soil has undergone dissipation of pore water pressure, and before this dissipation has occurred, because of the building up of pore water pressure, the particles were pushed away from each other. So, there were a lot of voids where particles were not there. When the pore pressure dissipation is over, the particles were in a very loose state, and then they will start settling. So, there will be settlement.

Lateral spread. There is a lot of lateral spread as a result because there was the development of pore water pressure. When this excess pore pressure started moving towards the drainage surface, that also took soil particles with it. So, you will see a lot of lateral spread, lateral flows also as a consequence of liquefaction occurrence.

Loss of support. If you are talking about foundations like a pile foundation is there or a shallow foundation is there and all the medium which was supporting the foundation from all around, that medium has actually been washed away. So, certainly, there will be a loss of support.

Same, you can see in terms of bridge piers also, so there will be a lot of, if liquefaction has occurred, then you will see there is no support laterally or from beneath. As a result, there will be settlement, failure, and tilting in the piers. Loss of lateral support, as I mentioned, even in terms of abutments also, you will see there is loss of lateral support; as a result, it will lead to failure. So many things are there which are basically the consequences one can witness at a particular site which has undergone liquefaction due to earthquake loading condition primarily. Otherwise, you have also seen there are sites which can undergo liquefaction due to monotonic loading condition.

Now there are different criteria based on which one need not go for detailed investigation, but even the soil type itself, moisture content, and properties of the soil itself will give you a fair idea about whether the soil is having susceptibility. So, that means if suitable loading condition is applied to the soil, that is a different part, but how about the soil? What is the property of the soil? Whether the property of the soil asserts that it can undergo liquefaction or not? So, that property will come under susceptibility. What is the susceptibility that a particular site can undergo liquefaction solely depends upon what are the soil properties, groundwater table, and subsequently we can also look into historical and geological criteria. So, historical criteria, one can refer to historical evidence to find out zones which have undergone liquefaction, and we can focus more on those locations which have undergone liquefaction in the past to understand the triggering mechanism and the in-situ characteristics of those particular sites. Geological criteria, one can also understand what are the conditions in which, what are the agencies which led to the deposition, what are the agencies which are, whether there was some cementation effect in that particular location or not. So, deposition, hydrological environment, and relative age of the soil deposit will also be studied in terms of geological criteria in order to understand the susceptibility of a particular site to liquefaction. Next is compositional criteria. What is the composition of in-situ soil available at a particular site?



So, Chinese criteria that was given by Wang in 1979, you can see here, so there is a plot of liquid limit on the y-axis and saturated moisture content on the x-axis, and then you can see potentially liquefiable soil and potentially non-liquefiable sites primarily based on the content of particles which are less than 0.005 mm. So, if the content of particles less than 0.005 mm is less than 20%, which was mentioned as the clay fraction, if it is 20%, and the plasticity index of the soil is less than 13, then you can call the particular soil potentially liquefiable. The region which can distinguish between potentially liquefiable and non-liquefiable also can be plotted using 0.87 times the liquid limit of the soil and then liquid limit equals 33.5% as shown in the first figure. In addition, if the natural moisture content of the soil is 0.87 times the liquid limit of the soil, or when the liquid limit of the soil is greater than 33%, you can call that the soil is potentially non-liquefiable. If the clay fraction is more than 20%, again referring to the particle size which is given for potentially liquefiable sites, that is 0.005 mm, you can call the soil potentially non-liquefiable, or when the plasticity index is greater than 13, you can call that the soil is potentially non-liquefiable.

Andrew and Martin (2000)

Potentially liquefiable soil
- Clay fraction (0.002mm) ≤ 10%
- LL ≤ 32%

Non-liquefiable soil
- LL>32%
- Clay fraction (0.002mm) >10%

Similarly, Andrew and Martin in 2000 proposed another criterion where you can say if the clay fraction, where primarily mentioned about 0.002 mm, that clay fraction is less than 10% and the liquid limit of the soil is less than 32%, such soil you can call as potentially liquefiable. Non-liquefiable sites, if the liquid limit is greater than 32% and the clay fraction is greater than 10%, you can call that based on the compositional criteria of the soil and moisture content, the soil may be liquefiable or non-liquefiable if it matches with the criteria mentioned on the screen.

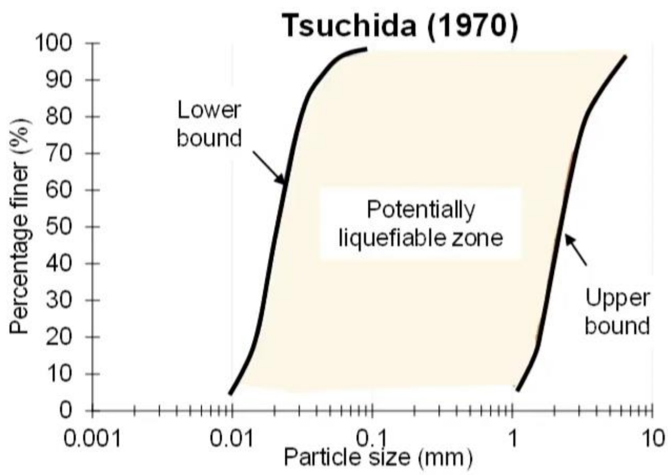


Fig redrawn based on Tsuchida (1970)

Then Tsuchida in 1970 also proposed the range in which potentially liquefiable sites are present, as can be seen in the third figure. This plot gives lower bound and upper bound particle size distribution curves. So, if you collect a sample from a particular site, obtain the grain size distribution of that sample, and try to put it over here, if that grain size distribution is falling between the lower range and the upper range, you can say in case favorable loading conditions are given, the soil compositionally has the possibility to undergo liquefaction, or there is susceptibility to undergo liquefaction if favorable loading conditions are given. So, there are three criteria based on which I mentioned over here, one can identify whether soil which is available in situ is potential to undergo liquefaction. Again, state criteria based on the initial state of the soil and how the initial state of the soil and subsequently the loading which will be applied, whether monotonic loading or cyclic loading, one can again bring into account the effective stress path and then see whether the soil is reaching its critical state and what is the path it is taking. Subsequently, we can decide whether it is undergoing flow liquefaction or cyclic mobility. Those we will discuss in coming lectures.

Again, there are different ways in which one can evaluate the liquefaction potential. As I mentioned in the beginning, there are different ways, but we will be focusing more on the cyclic stress approach. So, different ways which one can refer to are the cyclic stress approach, cyclic strain approach mostly followed, then we have the energy dissipation approach, we have the effective stress response-based approach, and then probabilistic liquefaction also is there where you can find out, considering different properties of the soil, what is the probability that it can undergo liquefaction or not. Further, we will continue with respect to the cyclic stress approach. Now in the cyclic stress approach, the main objective is to compare, based on the in-situ characteristics of the soil, how much is the cyclic resistance the soil can offer. Depending on the loading condition, which you can say maybe primarily because most likely what magnitude of the earthquake and what peak ground acceleration the surface will be experiencing during a particular earthquake, these two properties will help you in understanding how much is the cyclic loading or cyclic stress which will be actually mobilized in the soil because of a particular earthquake loading.

So, comparing the stress which is because of earthquake loading condition and resistance which is because of in-situ soil condition, we can understand whether for this particular stress condition and this particular resistance the soil is offering, whether the soil will undergo liquefaction or not. So, earthquake loading condition, earthquake-induced loading in terms of cyclic shear stress, as we mentioned, there will be cyclic loading which is happening because of the earthquake. In order to find out the liquefaction, this nature of cyclic loading is compared with the resistance which the soil is going to offer in terms of cyclic stress ratio. Cyclic stress ratio means whatever loading or cyclic shear stress which is being mobilized because of a particular earthquake loading condition in the soil medium, that will be compared with respect to the resistance the soil is offered. Depending upon the relative values of these two parameters, resistance and the loading condition, one can decide whether the soil is safe against liquefaction, whether the soil will undergo liquefaction, and subsequently it can be estimated.

Now here, one observation, when we talk about cyclic loading condition, primarily when we are dealing with the laboratory test like cyclic triaxial test, we will be subjecting that sample to cyclic loading with controlled amplitude and frequency content. In earthquake loading condition, because it is random in nature, the amplitude and frequency content are not the same, so it will be very difficult to find out what average frequency content and what average peak ground acceleration one should take into account in order to account for the pore pressure which will be generated due to actual earthquake loading condition. So, bringing those uncertainties into account in the cyclic stress approach, we will take another parameter which is called 0.65. This 0.65 has been proposed such that the maximum stress which is generating, maximum shear stress which is actually generating because of earthquake loading condition, can be approximated equal to 0.65 times the stress loading, the shear stresses which will be mobilized in the soil medium corresponding to cyclic loading condition. Now both of these will be used. So, this 0.65 is the conversion factor which will help in converting the irregular time histories because of actual earthquake loading to an equivalent series of uniform stress cycles. So, whatever earthquake loading has induced in the soil, you will see for some particular duration there will be peak amplitude, but other than that particular duration the amplitude of the vibration will be significantly lower. So, how to account for this peak as well as not the entire duration of the loading—the only the peak value was there. So, Seed and Idriss came up with this particular correlation that cyclic stress ratio can be correlated with respect to the maximum shear stress divided by the effective overburden pressure, multiplied by a function that is called a conversion factor from maximum to average cyclic loading, maximum during earthquake loading, average during cyclic loading or uniform number of cycles.

$$CSR = \frac{\tau_{cyc}}{\sigma_v'} = 0.65\frac{\tau_{max}}{\sigma_v'}$$

So, 0.65 was the factor which was given. Cyclic stress ratio (CSR) is cyclic stress ratio. It is given as the ratio of tau cyclic—how much cyclic loading is leading to cyclic shear stress—divided by effective overburden pressure. Now, this cyclic will be having uniform cycles of loading, so loading, then unloading, loading, unloading—those kinds of cycles will be there. So, when I say unloading, it is going—initially, it was like to motion; then, it will come to from motion. So, to and from motion with respect to mean positions—that will represent cyclic loading condition. Now, in this particular case, though it is cyclic, it is also uniform in nature. So, that is why it is more mentioned as uniform stress cycle. So, this is resembling uniform stress cycle, and this is resembling one characteristic of peak shear stress, which was mobilized in the soil medium due to actual earthquake loading. So, using this particular factor, 0.65 times tau max, we multiplied with respect to maximum shear stress by 0.65. Now, whatever we are getting, it is a cyclic value, which is representing the uniform cycle loading cycles in the soil medium.

Maximum horizontal acceleration
at the ground surface

$$CSR = 0.65 * \frac{\sigma_v}{\sigma_v'} * \frac{a_{max}}{g} * r_d$$

Shear stress reduction factor that accounts for the increase in confinement with depth.

So, this conversion factor, proposed for the multiplication function of 0.65, which was proposed, is used to convert, as per Seed and Idriss 1967, the irregular time histories of recorded ground motions to equivalent cyclic loading such that an increase in pore water pressure—so there is cyclic loading also, which is leading to building up of pore water pressure, and then irregular loading also, which is triggering to building up of pore water pressure. By using this conversion factor of 0.65 or multiplication function of 0.65 in both cases—whether you are talking about cyclic loading condition or you are talking about irregular ground motions—both are maintaining the development of pore water pressure uniformly. So, tau cyclic is used from tau max or a max—maximum shear stress, maximum ground motion, ground acceleration, or generally referred to as peak ground acceleration, which is the peak value of your acceleration times t record. Take that value, which is also a representation of maximum shear stress mobilized in the soil during actual earthquake loading condition, multiply it with respect to 0.65. So, you are actually going to get, corresponding to this value, how much is the cyclic component of a uniform number of cycles of loading. So, cyclic stress ratio, which is actually a way of determining how much loading is expected to be delivered because of earthquake loading condition in the soil medium—so cyclic stress ratio is 0.65 times. This is total stress at any depth of loading. Remember, cyclic stress ratio means liquefaction—actually development of pore water pressure. It is not only confined onto the ground surface but even at deeper depths.

Generally, it has been observed that even up to 20 meters depth, there are case histories where liquefaction has occurred. At a particular depth, the development of confinement—whether it is total stress or effective stress—is varying with respect to depth. This value will take maximum because maximum force, maximum loading condition, will be mobilized at the ground surface. So, a max at the ground surface, and then corresponding to that, how much increase or decrease will be there because of change in the confinement, that will be taken care of by a factor r_d, called stress reduction factor. So, 0.65 times total stress divided by effective stress at any depth of interest. So, if you are interested in finding out cyclic stress ratio at 2 meters depth, you will try determining the value of total and effective stress at 2 meters depth. r_d value—stress reduction factor—again at 2 meters depth. A max value will remain uniform throughout because it is the maximum value of peak ground acceleration or peak ground acceleration corresponding to the surface record normalized with respect to g. So, this equation is going to give you how much is the cyclic stress ratio generated by an earthquake, which actually generated or triggered peak ground acceleration of a max on the surface. Depending upon the depth at which you are determining the value of total and effective stress, you can actually determine how much is the development of cyclic shear stress because of the same earthquake loading at different depths. So, this way, we are able to determine how much earthquake loading is actually mobilized or how much shear stresses are actually being mobilized in the soil medium at different depths due to earthquake loading.

- **Stress-reduction factor (“r_d”)**

$$r_d = 1.0 - 0.00765z, \qquad for\ z \leq 9.15m$$

$$r_d = 1.174 - 0.0267z, \qquad for\ 9.15m < z \leq 23m$$

Liao and Whitman (1986)

$$r_d = 0.744 - 0.008z, \qquad for\ 23m < z < 30m$$

Robertson and Wride (1997)

$$r_d = \exp[\alpha(z) + \beta(z).M]$$

Idriss (1999)

$$\alpha(z) = -1.012 - 1.126\, sin\left(\frac{z}{11.73} + 5.133\right)$$

$$\beta(z) = 0.106 + 0.118\, sin\left(\frac{z}{11.28} + 5.142\right)$$

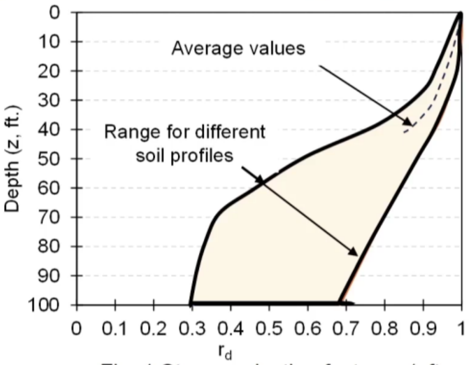


Fig. 1 Stress reduction factor, r_d (after Seed and Idriss, 1971)

The value of stress reduction factors, as we can see—because it is a representation of the confinement—you can see different researchers have given different correlations to find out what is the liquefaction potential and what is the stress reduction factor in a particular medium and how it is varying with respect to depth. So, z is basically the depth of the point of interest at which you are interested in finding out the value of cyclic stress ratio measured with respect to the ground surface. So, if you are going from 0 to 9.15 meters, you can see that stress reduction factor can be estimated from the first equation. If it is between 9.15 to 23 meters, you can go with the second equation. If it is between 23 and 30 meters, you can go with another equation proposed by another researcher. So, using the corresponding depth at which you are interested in finding out cyclic stress ratio, you can pick up the respective equation. I mean, I have mentioned here some equations—you can also search for many more equations which exist in the literature. Same way, Idriss in 1999, proposed this particular equation where the value of r_d is a function of alpha and beta, and subsequently, the alpha and beta are also related with respect to the depth measured from the ground surface to the point of interest where one is interested in determining cyclic stress ratio. So, based on this, you can find out the value of r_d. You will be having a bore log where the density of the medium is also given, and depth of the groundwater table is also given. So, using these two parameters—density variation with respect to depth and depth of groundwater table—we can determine the value of total stress as well as effective stress. r_d value—we have just seen how we can determine. a max value, one will be told, or based on your hazard analysis, one can estimate how much is the surface value of the ground acceleration. Take that value as a max. You can determine the value of cyclic stress ratio.

Now, one observation here—when we are talking about cyclic stress ratio, it is basically corresponding to the actual loading of an earthquake which actually triggered shearing shear stress in the medium. Now, this is going to give you the loading which is actually mobilized in the soil medium. We are interested in finding out the potential of a site to undergo liquefaction. So, in order to decide whether the soil is potential to undergo liquefaction or not, in addition to loading condition, there should be a mechanism through which one can determine how much is the in-situ shear strength. So, that in situ shear strength one can determine based on a number of methods—you can go with cyclic simple shear test, cyclic triaxial test. So, this is basically helping in understanding how much the in-situ strength of the soil medium is.

$$CRR_{M,\sigma'_v} = CRR_{M=7.5,\sigma'_{v=1atm}} . MSF . K_\alpha$$

Now, there is another term which will come into the picture—it is cyclic resistance ratio or CRR value. So, one can determine the value of CRR for any magnitude of interest and any depth of interest because resistance of the soil is actually changing because the medium itself is changing with respect to depth. So, corresponding to the unknown value of confining pressure or effective stress—this is. Now, you see here this equation—CRR. The equation of CRR was actually proposed—this particular equation—based on which one can determine the value of CRR, that is, cyclic resistance ratio. The equation was given based on the in-situ measurement of SPT values, primarily corresponding to 7.5 magnitude earthquakes. So, that's why the equation of CRR—it is primarily given corresponding to (CRR) suffix M equals to 7.5. That means this particular equation will give you the value of cyclic resistance ratio or measure of in situ resistance which the soil is going to offer corresponding to a magnitude of 7.5. That means this particular equation will give you the value of the cyclic resistance ratio or measure of in-situ resistance, which the soil is going to offer corresponding to a magnitude of 7.5. CSR was not given in terms of a particular magnitude; it was corresponding to actual earthquake loading. Here, it is giving corresponding to a magnitude of 7.5 and also with respect to atmospheric pressure of 1 atm. So, again, here the cyclic resistance ratio is a function of this particular empirical equation, which was given by Idriss and Boulanger in 2010.

$$CRR_{M=7.5,\sigma'_{v=1atm}} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right)$$

Idriss and Boulanger (2010)

You can see the cyclic resistance ratio is a function of one parameter, which is called $(N_1)_{60CS}$. Now, here the term $(N_1)_{60CS}$ means n is the SPT N-value, standard penetration test N-value, at a particular depth of interest, normalized for 1 atmospheric pressure, normalized for one atmospheric pressure corresponding to 60% hammer energy, and corresponding to a clean sand condition. One atmospheric pressure, again, you have standardized clean sand condition because the methodology was primarily given for clean sand conditions. So, anything which is deviating the soil properties with respect to clean sand conditions, there will be suitable empirical correlations which will be applied or corrections which will be applied. Finally, once you get the value of SPT corresponding to 60% hammer energy, clean sand condition—so CS is clean sand condition—and N_1 is 1 atmospheric pressure, this value of SPT N, which is modified, will be put over here in this particular correlation, and you can find out how much is the value of the cyclic resistance ratio of the soil medium at a particular depth corresponding to one atmospheric pressure and a magnitude of 7.5. Now, one question which may arise is that not every time the earthquake which is going to trigger liquefaction at a particular site is equal to 7.5. So, what to do in that particular case? Because your CSR value, cyclic stress ratio, is basically corresponding to actual earthquake loading, but the cyclic resistance ratio is, so far, corresponding to a magnitude of 7.5.

$$CRR_{M=7.5,\sigma'_v=1atm} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10.(N_1)_{60}+45]^2} - \frac{1}{200}$$

FC= 5%

Seed et al. (1984)

Where, $(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$ Idriss and Boulanger (2010)

Before we will go to 7.5, you will see that $(N_1)_{60}$ is basically the value of SPT for one atmospheric pressure and 60% hammer energy—that is called as if that value is available to you. And if the fine content, which is primarily the clay content and some portion of silt, is available, if the fine content which is available in your soil medium is less than 5%, then you say the fine content will not have much effect on deciding the liquefaction potential of your site. You can go ahead with this particular part. Now, here there is no mention of CS because when the fine content is less than 5%, it will not have an effect on controlling the liquefaction potential. So, you consider a soil which is having fine content less than 5% as equal to a clean sand condition. So, in this particular case, whether you go with $(N_1)_{60CS}$ or you go with $(N_1)_{60}$, both are going to give you the same value of the cyclic resistance ratio, or both are going to offer the same resistance to external loading conditions. So, in this particular case, again, you are getting the same value of CRR, that is, at magnitude 7.5 and one atmospheric pressure. Determine the value if the fine content is less than 5%. So, based on this, you can find out $(N_1)_{60CS}$. Otherwise, if you are interested in general, $(N_1)_{60}$ plus delta $(N_1)_{60}$. So, delta $(N_1)_{60}$ is basically the correction due to fine content.

$$\Delta(N_1)_{60} = \left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right)$$

So, if you know the fine content, you can put it in this particular equation. Fine content is given in terms of percentage; put it in this particular equation, and you will get the value of delta $(N_1)_{60}$. That is the corrected n-value for corresponding to fine content. So, delta $(N_1)_{60}$, you bring it here, put it over here, this value $(N_1)_{60C}$. $(N_1)_{60}$ is already known corresponding to 60% hammer energy and one atmospheric pressure. Add up these two, and you will get the value of SPT N corresponding to clean sand condition because now you have applied correction due to fine content. So, the equation which was given at the top was given by Seed et al. in 1984, and then further other equations are also given.

$(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$ Seed and Idriss (1971)

α=0	for FC ≤ 5%	$\beta = 1.0$	for FC ≤ 5%
$\alpha = \exp[1.76 - \left(\frac{190}{FC^2}\right)]$	for 5% < FC < 35%	$\beta = [0.99 + \left(\frac{FC^{1.5}}{1000}\right)]$	for 5% < FC < 35%
α=5.0	for FC ≥ 35%	$\beta = 1.2$	for FC ≥ 35%

So, here you can see $(N_1)_{60CS}$, Seed and Idriss 1971. You can see over here this particular equation, alpha plus beta $(N_1)_{60}$. So, this particular equation, again, depending upon the value of fine content, you can have the value of alpha and beta. So, three primary divisions are there: fine content less than 5%, fine content between 5% and 35%, and if fine content is more than 35%, then further increase in fine content, whether it is 36% or whether it is 46%, is not going to make much difference in terms of the cyclic resistance ratio. It will offer more or less the same value.

$$(N_1)_{60} = N_m . C_N . C_E . C_B . C_R . C_S$$

Where, N_m= Measured standard penetration resistance,
C_N= Overburden correction factor,
C_E= Correction for hammer energy ratio,
C_B= Correction for borehole diameter,
C_R= Correction for rod length,
C_S= Correction for samplers with or without liners.

So, again, you had some value. So, $(N_1)_{60}$ means field-measured SPT N-value corrected to field measurements. As I mentioned, SPT N-value—if you go deeper and deeper, there will be an effect because of confinement. Similarly, because of the change in the diameter of the borehole, there will be some effect on the SPT value. Because of rod length, the rod through which you are actually transferring the hammer impact to your soil sample, again, rod length correction will be there. Then liner correction will be there. Many a time, you provide a liner in the borehole such that the borehole should not undergo collapse. Hammer energy correction should be there, overburden correction—so these are all parameters which are mentioned over here. This is the field-measured N_m, which is measured at different depths. Now, there are other parameters which can actually contribute to your SPT N-value. That means overburden pressure, hammer energy—as we mentioned, there will be some hammer which will be actually transferring the impact. So, what is the efficiency of that particular hammer? Then borehole diameter, rod length correction, and liner correction. So, all these are basically the corrections which will actually change the field-measured SPT N-value. And accordingly, these corrections will be applied to the field-measured SPT value (N_m). You just product with this, and you will get the value of $(N_1)_{60}$, which was mentioned in the previous slide. So, this is the $(N_1)_{60}$ which you are going to get. In addition, you will have some component which is coming from the fine content. So, bring that component over here; then you will have $(N_1)_{60CS}$.

$$C_N = \left(\frac{P_a}{\sigma_v'}\right)^{0.5} \leq 1.7$$

Liao and Whitman (1986)

$$C_N = \left(\frac{2.2}{1.2 + \sigma_v'/P_a}\right) \leq 1.7$$

Kayen et al. (1992)

$$C_N = \left(\frac{P_a}{\sigma_v'}\right)^{m} \leq 1.7$$

$$m = 0.784 - 0.0768\sqrt{(N_1)_{60cs}}$$

Idriss and Boulanger (2010)

P_a= Atmospheric pressure of 1 atm (100 kN/m^2)

So, there are different correlations which are going to find out the overburden correction, as I mentioned on the screen, by different researchers. So, one can refer to these correlations and the upper bound to this particular correlation. In addition to this, there are charts and tables which are given, referring to which you can find out. Like, for rod length correction, depending upon what is the length of the rod or which depth particularly you are referring to, you can pick up the absolute value of rod length correction. Similarly, the borehole diameter correction—because at different places, you can see different diameters of boreholes are being used to drill a borehole while measuring the SPT N-value.

Table 1 Values of C_R suggested by various researchers

Rod length (m)	Youd et al (2001)	Skepmton (1986)	Robertsen and Wride (1997)	Bowles (1996)	Seed et al. (1984)
<3	0.75	0.75	-	0.75	0.75
3-4	0.80	0.75	0.75	0.75	1.0
4-6	0.85	0.85	0.85	0.85	1.0
6-10	0.95	0.95	0.95	0.95	1.0
10-30	1.00	1.0	1.0	1.0	1.0

Table 2 Values of C_B suggested by various researchers

Borehole diameter (mm)	Skepmton (1986)	Robertsen and Wride (1997)	Bowles (1996)
60-120	1	1	1
150	1.05	1.05	1.05
200	1.15	1.15	1.15

Table 3 Values of C_S suggested by various researchers

Presence of liner	Skepmton (1986)	Robertsen and Wride (1997)	Bowles (1996)
No liner	1.2	1.1-1.3	1
Liner: Dense sand	1.0	1.0	0.9
Liner: Loose sand	1.0	1.0	0.8

So, again, you can refer to the literature which is given over here, find out the borehole diameter correction. Then liner presence—if it is there or it is not there, then there are different researchers who have proposed different correlations. One can refer to these charts, get the value of C_B, C_R, C_S. Refer to hammer energy, then find out how much is the hammer energy correction, overburden correction, which is given in the previous slide. So, using all these corrections and the field-measured SPT value, one can determine the value of $(N_1)_{60}$. Take out the bore log from where you can actually find out what is the fine content in the soil medium. Depending upon the percentage of fine content, you determine the value of delta $(N_1)_{60}$. Club $(N_1)_{60}$ and delta $(N_1)_{60}$, and you find out the value of $(N_1)_{60CS}$. Put this value of $(N_1)_{60CS}$ in the correlations which are given for CRR, and you will get the value of CRR at 7.5 magnitude and one atmospheric pressure.

Overburden correction factor (“K_σ”)

$$K_\sigma = 1 - C_\sigma \left(\frac{\sigma_v'}{P_a}\right) \leq 1.1$$

$$C_\sigma = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60cs}}} \leq 0.3$$ Boulanger (2003)

Magnitude scaling factor (MSF)

$$MSF = 6.9\, exp\left(\frac{-M}{4}\right) - 0.058 \leq 1.8; \quad \text{for sand}$$

$$MSF = 1.12\, exp\left(\frac{-M}{4}\right) + 0.828 \leq 1.13; \quad \text{for clay}$$

Boulanger and Idriss (2007)

$$MSF = \frac{10^{2.24}}{M_W^{2.56}}$$ Seed (1982, revised)

So again, an additional factor which will actually bring the change between 7.5 magnitude to any other magnitude at a particular site of interest which you have found out based on your hazard analysis. So, the magnitude scaling factor will take this additional factor. Because of magnitude scaling, this particular factor is the magnitude scaling factor. This magnitude scaling factor is actually going to tell you if any magnitude, which is greater than 7.5, will have an effect on the duration of motion. So, you will apply an additional magnitude scaling factor, which can be determined using the value of M. Usually, it is referred to as the magnitude corresponding to which we are interested in finding out the liquefaction potential. You can put it over here. So, different correlations are given by different researchers. You can determine the value of magnitude scaling factors.

$$FOS = \left(\frac{CRR_{7.5}}{CSR}\right) * MSF * K_{\sigma}$$

Once the value of the magnitude scaling factor is known to us, the CRR corresponding to 7.5 can be compared with respect to the cyclic stress ratio. So, CRR into 7.5 is going to actually help in determining the resistance the soil is going to offer corresponding to the actual magnitude. And then one can compare it with respect to the cyclic stress ratio. So, factor of safety is equal to CRR at 7.5 over CSR, multiplied by the magnitude scaling factor. An additional factor can be there because of the overburden effect. And then one can determine what is the factor of safety with respect to liquefaction, or for a particular earthquake loading condition, what is the factor of safety.

Numerical problem.1 Calculate the liquefaction potential of a soil profile (in terms of FOS at various depth) based on the borehole data provided below using Seed et al. (1984) procedure.

Depth (m)	Density (kN/m³)	Measured SPT	C_E	C_B	C_R	C_S	Fine Content (%)
1.85	17.85	12	0.7	1.05	0.75	1	89
3.35	18.64	14	0.7	1.05	0.75	1	50
4.85	17.46	14	0.7	1.05	0.75	1	4
6.35	18.64	15	0.7	1.05	0.75	1	3
7.85	18.93	29	0.7	1.05	0.75	1	39
9.35	18.74	25	0.7	1.05	0.75	1	75

Assume water table at ground surface. Use Liao and Whitman (1986) formula to determine C_N. $K_{\sigma} = 1$. $\frac{a_{max}}{g} = 0.36$, $r_d = 1.0 - 0.00765z$

Use, $MSF = \frac{10^{2.24}}{M_W^{2.56}}$

So, a numerical example has been given over here: calculate the liquefaction potential of a site. Based on the bore log, the in-situ densities are given, and the field-measured SPT N (so you can call it as Nm-value) is given. These two values are given. Then hammer energy, borehole diameter—these things are already given. So, you can refer to these and try determining. Fine content is also given to you. So, this is the fine content, or FC value, directly given in terms of percentage. So, depending upon the value of fine content, we can refer to whether it is less than 5%, 5% to 35%, or greater than 35%. Pick up the correlation, determine how much is the value of delta $(N_1)_{60}$. In addition, we have been given where k-sigma equals 1. C_N one can refer to Liao and Whitman (1986). So, already some guidelines are given based on which correlation one has to use and determine the value of different coefficients. And the correlation for determining the magnitude scaling factor is also mentioned over here. The correlation with respect to stress reduction factor determination is also mentioned. So, all the correlations or the value of the different corrections are also mentioned over here.

Solution:

Depth (m)	Density (kN/m^3)	Measured SPT	C_E	C_B	C_R	C_S	Fine Content (%)	C_N	α	β	$(N_1)_{60cs}$	CRR 7.5	MSF	rd	CSR	FOS= $CRR_{7.5}$ *MSF/ CSR
1.85	17.85	12	0.7	1.05	0.75	1	89	1.70	5	1.2	18.49	0.20	1.44	0.99	0.51	0.56
3.35	18.64	14	0.7	1.05	0.75	1	50	1.70	5	1.2	20.74	0.22	1.44	0.97	0.49	0.66
4.85	17.46	14	0.7	1.05	0.75	1	4	1.59	0	1	12.26	0.13	1.44	0.96	0.50	0.39
6.35	18.64	15	0.7	1.05	0.75	1	3	1.38	0	1	11.38	0.13	1.44	0.95	0.49	0.37
7.85	18.93	29	0.7	1.05	0.75	1	39	1.23	5	1.2	28.52	0.39	1.44	0.94	0.47	1.18
9.35	18.74	25	0.7	1.05	0.75	1	75	1.12	5	1.2	23.50	0.26	1.44	0.92	0.46	0.82

What we will do is try determining based on the densities, field SPT measures, borehole diameter, hammer energy correction. So, hammer energy in this particular case is mentioned as 70% hammer energy. So, we are using it as 0.7. Then borehole diameter, rod length correction, and fine content correction. Refer to whatever information is given. If it is not given, we can clearly mention referring to which particular literature chart what values we are referring to. Then fine content corrections are also there, overburden correction already—one equation is given. So, referring to that, one can find out the value of alpha and beta, and determine the value of $(N_1)_{60CS}$. Using this value, you can determine the value of CRR. Now, the magnitude is already given. The correlation is also highlighted. So, one can determine the value of the magnitude scaling factor. The correlation to use for the stress reduction factor is also highlighted. So, you can determine the value of RD. Based on the a-max value which is given over there, one can determine the value of cyclic stress ratio. So, using this, you can see in the last column. Now, one observation here: generally, when we are determining the factor of safety for a particular site, while determining the total and effective stress, we refer to the groundwater table being located at the ground surface. This is primarily considered so that the worst-case scenario with respect to liquefaction triggering can be taken into account.

So, using those calculations, we have determined the value of the factor of safety at different depths. Now, referring to a particular location, we will call that particular location potentially liquefiable, even if one stratification between that particular location is undergoing liquefaction. So, here, we can see, except for one location which has a factor of safety of 1.18, all other locations are having a factor of safety very low. That means all these locations are potentially liquefiable. This is the boundary condition, almost close to 1. But even if any one layer would have shown a factor of safety less than 1, we would have said that this particular site is potentially liquefiable. Because even if one layer undergoes liquefaction, that can lead to disturbance to the entire soil column. So, we will not say that one particular layer is liquefiable or not. Once we are asked to determine the potential for liquefaction at that particular site, we will try to find out based on the minimum factor of safety throughout the length of the borehole. If it is less than 1 or generally 1.5, we call that particular site potentially liquefiable.

Okay, so one can practice these numericals, and that will give confidence about how to go ahead with the calculation for liquefaction potential. As I mentioned, this lecture (Lecture 24) is primarily to understand how to find out the liquefaction potential, what the triggering mechanism is, what the state criteria say about liquefaction, and how one can differentiate between flow liquefaction and cyclic mobility. We can discuss those in upcoming lectures. So, thank you, everyone. Thank you.