Soil Dynamics
Prof. Deepankar Choudhury
Department of Civil Engineering
Indian Institute of Technology, Bombay

Module - 4
Dynamic Soil Properties
Lecture - 22
Liquefaction, Preliminary sceening,
Simplified Procedure for Liquefaction

Let us continue with our lecture on soil dynamics of module four, that is on dynamic soil properties.

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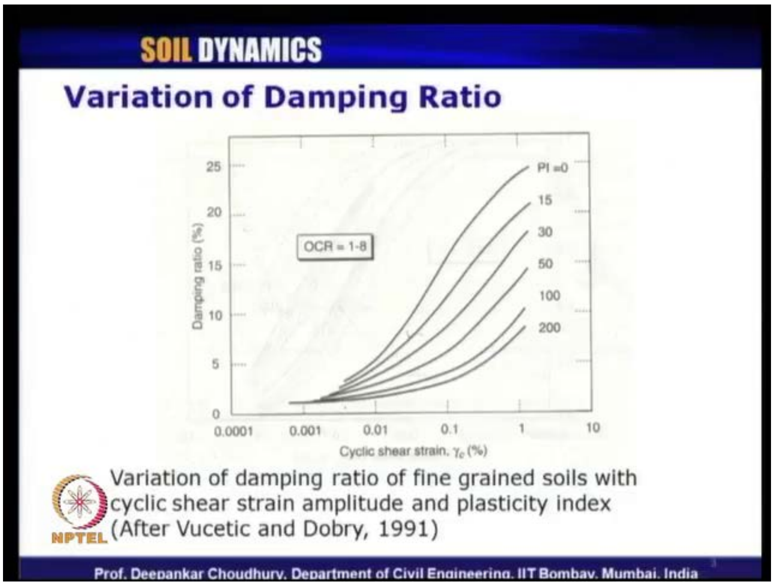

So, a quick recap of what we have studied in the previous lecture. We have seen the variation of damping ratio with different values of cyclic shear strain and for different plasticity index for fine grained soil as mentioned by Vucetic and Dobry in 1991. And in that curve we mentioned that P I equals to 0 curve; we should not use.

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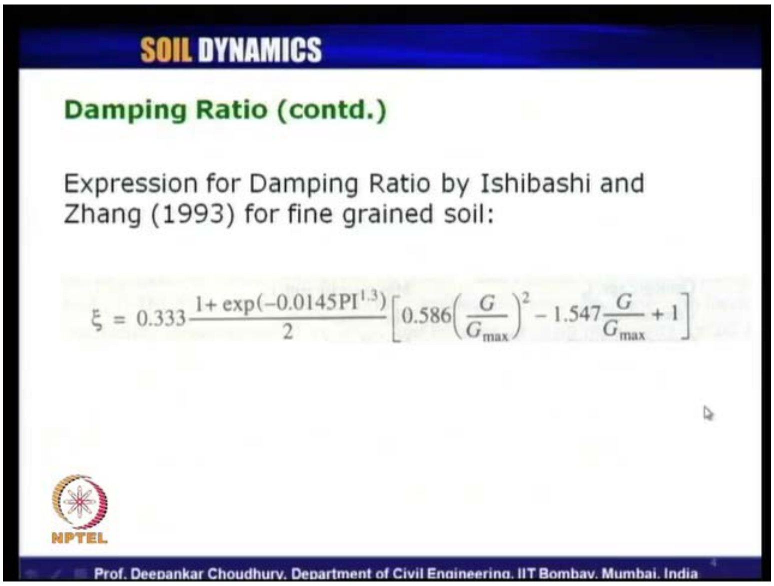

Also, the empirical relation to compute the damping ratio of any fine grained soil as proposed by Ishibashi and Zhang in 1993 is given by this expression; it is a function of g by g max value plasticity index value.

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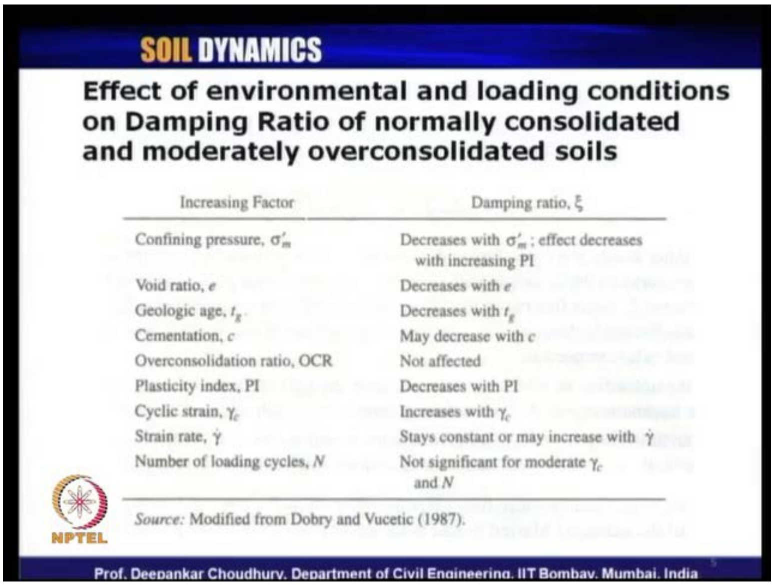

Also the effect of various environmental and loading conditions on the values of damping ratio for normally consolidated and moderately over consolidated soils we have seen. That is with increasing factors of these parameters how the damping ratio are changing? This was mentioned by Dobry and Vucetic in 1987.

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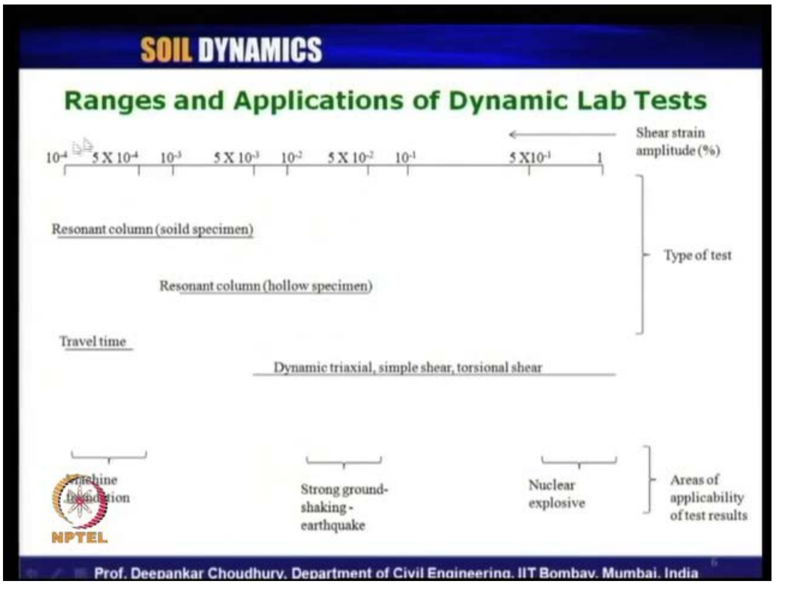

This was one of the most important chart, which we have mentioned. That it shows us the ranges and applications of various dynamic laboratory test which we should use like resonant column test with solid specimen or hollow specimen, travel time method dynamic or cyclic triaxial test, cyclic simple shear test, cyclic torsional shear test based on the different values of their ranges of shear strain amplitude. And that decides us their areas of applicability whether it is for nuclear explosive; it will be at high shear strain amplitude. If it is strong ground-shaking-earthquake, it is intermediate shear strain amplitude. And if it is for the case for machine foundation, it will be at very low shear strain amplitude. So, depending on that what test should be useful for what kind of design, we have to find out from this chart.

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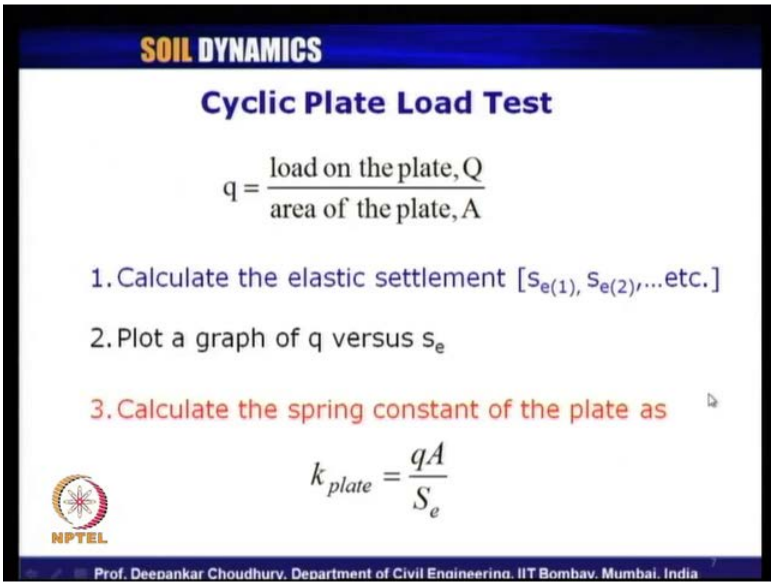

Also, we have gone through a cyclic plate load test. From cyclic plate load test, we can obtain the spring constant.

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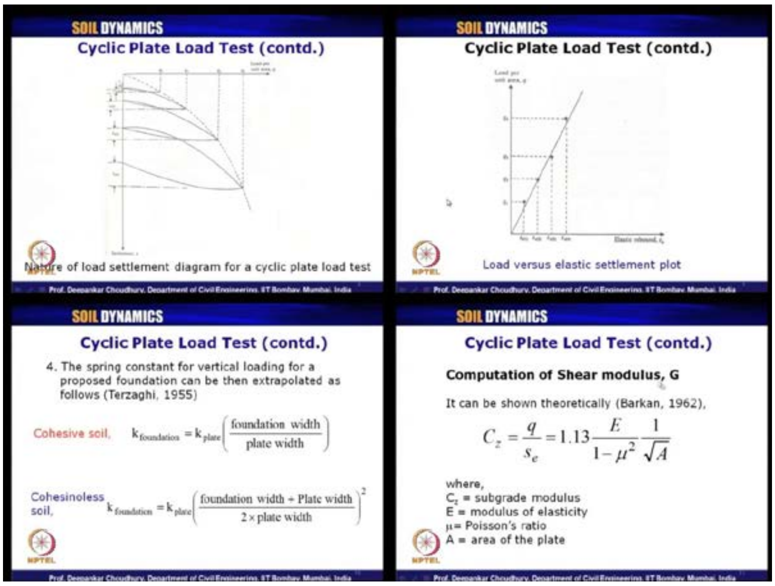

For the plate from the load versus settlement curves in cyclic fashion by computing the elastic component of the settlement. So, if we plot the load versus elastic settlement, then we will get linear relationship and the slope of that line will give us nothing but the dynamic spring modulus. For cohesive soil the K of foundation we can compute by using this relationship, once the K of plate is obtained and for cohesionless soil also K of foundation can be obtained by using this expression. How to compute the shear modulus G? Barkan has proposed this relationship were C z is subgrade modulus; dynamic sub grade modulus for vertical vibration only, as I have already mentioned in the previous lecture. E is modulus of rigidity or modulus of elasticity, mu is Poisson's ratio and A is area of the plate.

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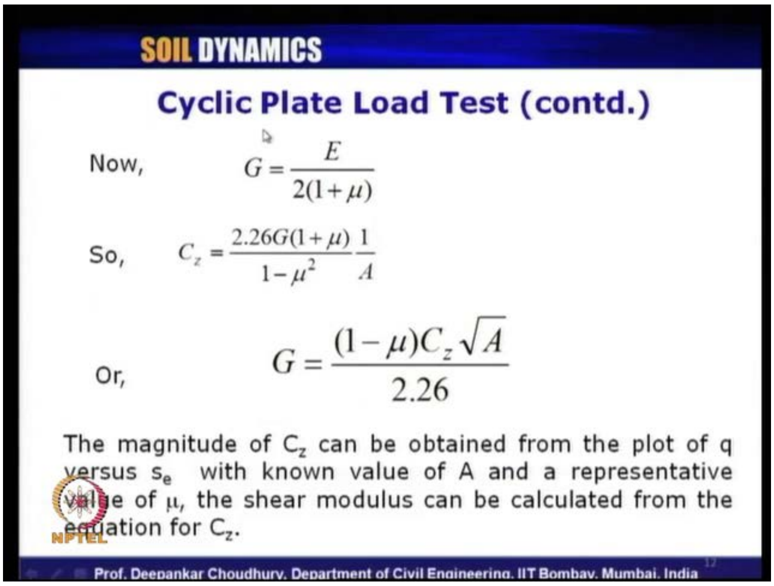

And, G and E are correlated between them with respect to mu like this. So, G can be expressed in terms of C z by this expression.

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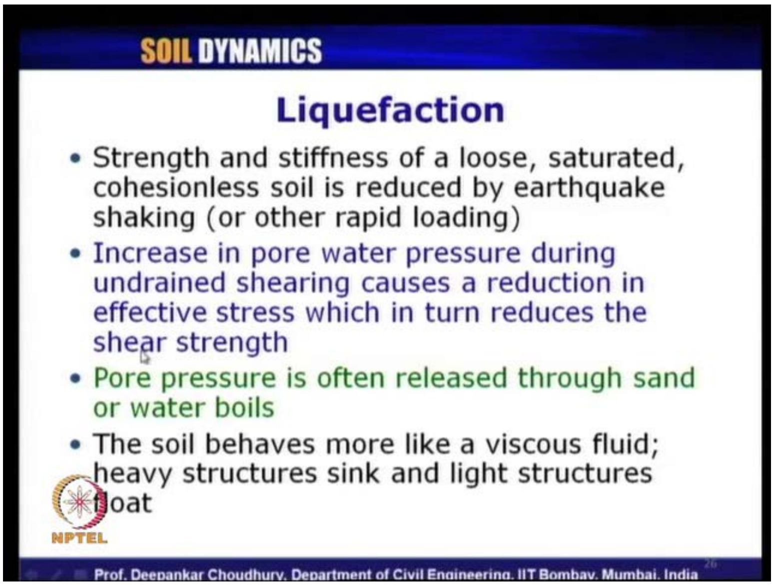

Now, let us start with our next subtopic which is liquefaction. What is liquefaction? Technically speaking it is nothing but strength and stiffness of a loose saturated cohesionless soil is reduced by earthquake shaking or by any other kind of rapid loading. That reduction in the strength and stiffness in the loose saturated cohesionless soil is referred as liquefaction. What happens during the liquefaction? Increase in the pore water pressure during undrained shearing; it causes a reduction in the effective stress which in turn reduces the shear strength. Because due to application of the dynamic loading like earthquake shaking or any other dynamic load, the shearing occurs and it is so rapid because dynamic loads are so rapid; it is the condition of undrained condition. So, that is why it is another important point you should note. In most of the analysis we have to use undrained parameters of the soil testing rather than drained parameters.

So, because of this undrained condition what happens during the process of shearing due to this dynamic loading? Pore pressure keep on increasing. As the pore pressure gets developed, we know the Terzaghi's relationship of the effective stress. Obviously, the effective stress will keep on decreasing because total stress remains same. If effective stress decreases, the shear strength is a function of effective stress only. So, obviously the shear strength of the soil also decreases. So, that is the phenomenon of liquefaction.

Pore pressure is often released through sand or water boils. Finally, when the pore pressure keeps on increasing, when they equate with respect to total stress; in that situation your effective stress will become 0. And that means your soil skeleton, solid material is not capable of taking any load. And if entire load is governed by the fluid material or water; water cannot take any shearing load. So, what will happen? Finally, it will burst out; means the water will released or pore pressure will get released in the form of soil as if it has no solid material, but as if entire thing is flowing as a fluid material. So, that is why it comes through as a sand or water boils condition. And the soil behaves more like a viscous fluid in that case. So, obviously heavy structures will sink down and if it is a light structure; it may float.

So, it is not necessarily that all types of structure will get same way affected by the concept of liquefaction, it depends on what type of structure you are considering, their frequency, natural frequency, what is the applied frequency during the earthquake and so many other factors are involved in it.

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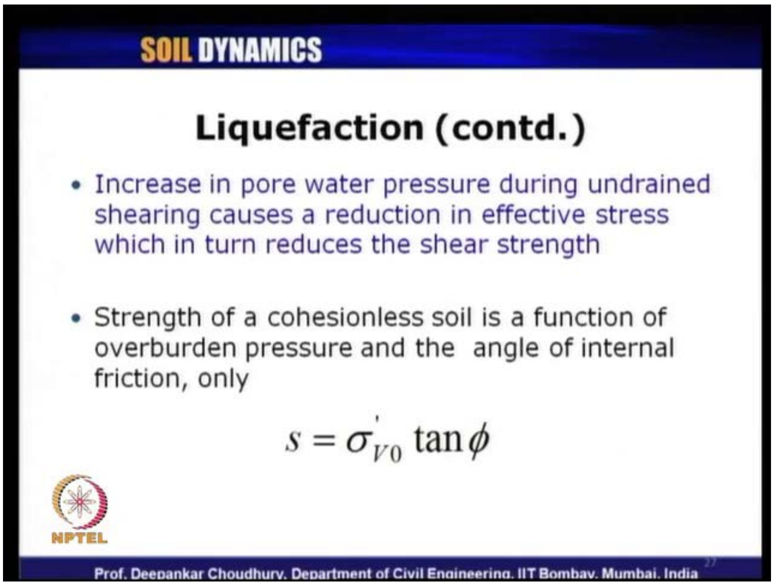

So, as we have mentioned in the liquefaction case increase in pore water pressure during the undrained shearing, it causes a reduction in effective stress which in turn reduces the shear strength. So, the shear strength expression for a cohesionless soil is given by this one; this well known expression of Mohr coulomb expression, right. The considering the Mohr coulomb failure criteria and Terzaghi’s shear stress equation.

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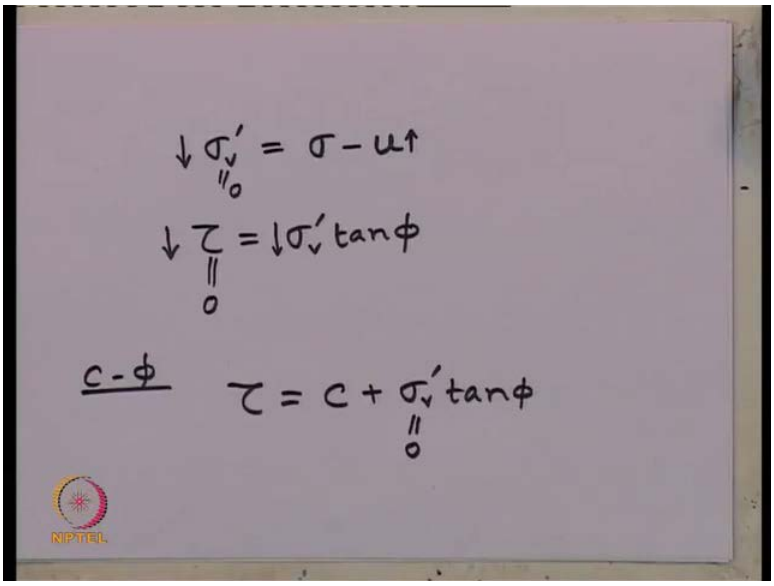

What we know that effective shear stress is nothing but total stress minus pore water pressure. And in case of liquefaction, what we said? This u value is keep on increasing.

Ultimately one situation will come; when it equates the total stress, in that case your effective stress become 0.

If effective stress become 0, what happens in your Mohr coulomb equation. Mohr coulomb failure equation is nothing but for a cohesionless soil is given by shear strength is effective stress times tan of phi; phi is the friction angle of the soil. So, during the liquefaction process we said this pore pressure increases, that is why this effective stress decreases, effective stress decreases means the shear strength of the soil decreases; that is the phenomenon of liquefaction. And full liquefaction or complete liquefaction means, it becomes 0; in that case this effectively becomes 0. So, there is no shear strength available in the soil and that is why it flows like a fluid; because fluid cannot take any shear. So, entire material flows as a as if it is a fluid material. So, this is about cohesionless soil.

So, I would like to draw your attention once again, these terminologies. It should be a loose and it should be saturated condition; so that water pressure is available fully, it should be cohesionless soil. If it is not so, why these three conditions are required? Let us look back here in this sheet. First of all if it is not saturated, let us look at here. So, what happens if it is not a cohesionless soil; if it is a c phi soil, for a c phi soil what is the expression for your shear strength? Tau equals to c plus sigma v dash tan phi. So, during the process of liquefaction, even if this value goes to 0 but still your soil is some having some shear strength, because of the presence of the cohesion component. So, that is why it is mentioned that liquefaction is a technically phenomenon which is valid for cohesionless soil.

But does it mean that c phi soil or cohesive soil cannot liquefy? Yes, they can also behave like same way as liquefaction, but there the technical term is different. Let me show you what happens. Suppose, if it is a c phi soil or c soil, so this component is present. So, shear strength truly speaking it is not going to be 0, but in that kind of soil as this effective stress is not present; it will behave as if your pour water pressure is keep on increasing. So, finally, your water will dominate in the soil media; it will be something like in a beaker or in a jar. You have full of water inside that you have some clay material or c phi soil material.

So, the behavior of it will be though it is having a cohesion component, but it cannot take any load; because it is dominated by the pour pressure or water generated inside it. So, it will be something like entire clay material is pumped out or burst out because pour pressure gets to be generated after a certain time, it it in keep on increasing and becoming huge. So, that phenomenon is commonly called as technically clay pumping or mud pumping.

So, clay pumping and mud pumping is a similar phenomenon of liquefaction but truly speaking, if we say technically liquefaction is for cohesionless type of soil or for clayey soil or c phi soil. If we find similar kind of phenomenon, where your soil is flowing like a fluid; we can term it as clay pumping or mud pumping also. But manier times we generalize and we mentioned that also as liquefaction. But you understood the basic concept of theory; that is how for a sandy material and how for a clayey material and c phi material the concept changes, though the behavior or final output remains same.

Now, thus let me concentrate on the another term which I have mentioned; for liquefaction we need a saturated condition. If the soil is not saturated does it mean that it cannot be subjected to a liquefaction? Liquefaction means it is flowing like a fluid; that we have generalized. So, can a dry soil flow like a fluid? If we pose this question to you, let me explain here through this picture.

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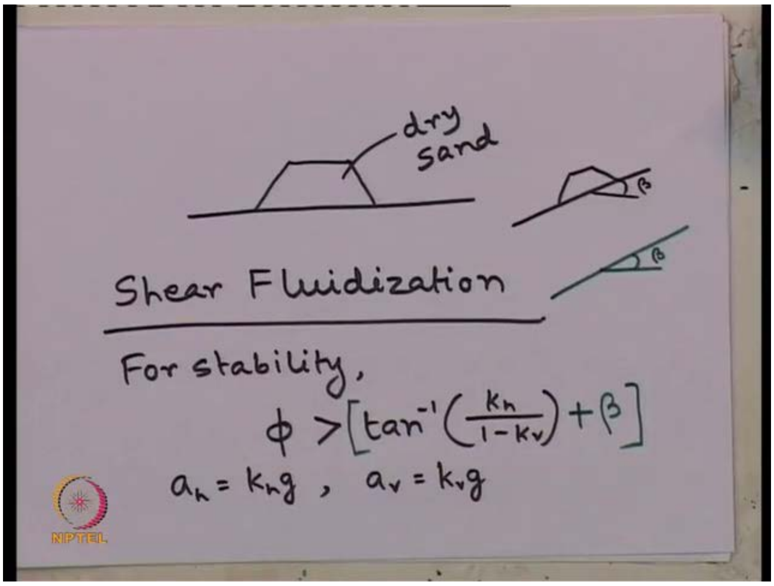

Suppose, you have taken dry sand, let us say dry sand on top of a board or on top of a table. And you heat it up, so it will remain stable up to a certain slope and up to a certain height; based on it is angle of repose. Just you put a paper or table on top of it, you keep on dumping the dry sand; it can stand freely up to a particular height, fine. The slope angle will be it is angle of repose. Now, what you do you start shaking this one; you start shaking your table, slowly. After certain time if you do the shaking vigorously, what will happen to your soil? It will flow out, means it will move down; it will flow in the entire tray or entire plate or entire paper.

Why it is happening? How does it happening? The phenomenon is known as shear fluidization, this is called. So, in case of shear fluidization for stability against shear fluidization, the condition is phi should be greater than tan inverse k h by 1 minus k v; where this a h is horizontal acceleration expressed by k h times g and a v is vertical acceleration of dynamic load. Suppose, if it is earthquake load a h is horizontal seismic acceleration, a v is vertical seismic acceleration and g is acceleration due to gravity. So, these are coefficient; this is called horizontal seismic acceleration coefficient k h, this is called vertical seismic acceleration coefficient k v.

So, if we your soil has to be stable under certain combinations of earthquake acceleration like this, the phi value of the soil must be greater than tan inverse of this. This is because so that no shear fluidization occurs in the sample. And if it is given in a slopping ground with an angle beta, in that case for stability the equation will change to; so let me draw it once again. If we have a slopping ground beta with respect to horizontal, for stability this should be the expression; the phi should be greater than beta plus tan inverse k h by 1 minus k v, to avoid shear fluidization.

So, in this case also what we have seen, for the case of completely dry sand if we are shaking the table or paper or plate on top of which we have taken that sand it can also flow like a fluid. It can behave also like a fluid is flowing; though it is a completely dry sand, there is no presence of water. That phenomenon is called shear fluidization which is similar to the concept of liquefaction, but technically they are different. So, many layman or general people they call all these together, that is whenever the soil behaves as a fluid; they term it as a liquefaction. But now we know after studying this soil dynamics that the behavior of these three different cases or three different incidents are physically different.

So, one is true liquefaction another is clay pumping or mud pumping and the other one is shear fluidization. So, that is why in the definition it is mentioned the for liquefaction loose saturated cohesionless soil; these are the conditions required. But other soils also can have a similar state of liquefaction; that is it can also flow under application of earthquake or any kind of rapid dynamic loading flow like a fluid.

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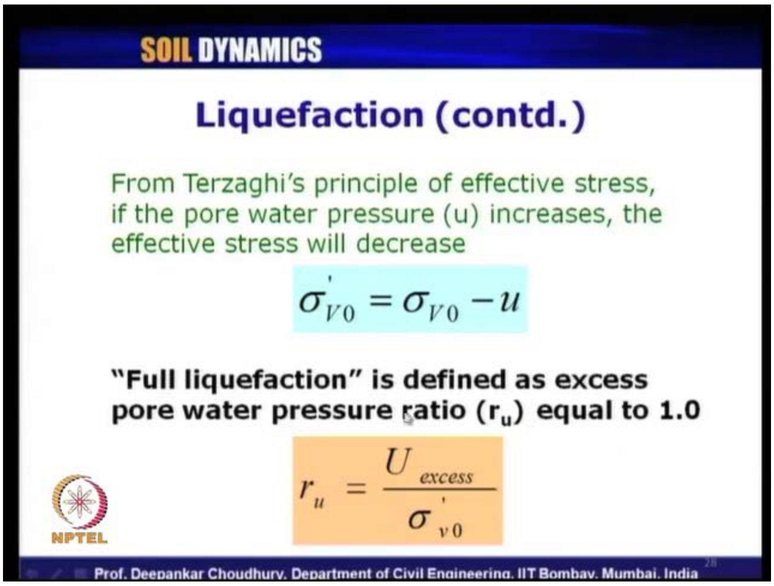

So, what I have explained just now, once again here you can see the Terzaghi's principle of effective stress. We know the effective stress equation is total stress minus pore water pressure. And the term full liquefaction is used when the excess pore water pressure ratio, it is given by generally the symbol r u. r u is give as excess pore pressure generated divided by the effective stress of the soil. When this ratio becomes equal to 1 that means the full liquefaction has occurred. Otherwise, it will be partial liquefaction if it is less than 1 and greater than 1 is of course, does mean full liquefaction has occurred and now it is fully flowing like a fluid. So, that is the basic estimation or computation or identification of a whether a soil has liquefied or not fully or partially through these expressions.

Now, let us see what are the various factors which influence the liquefaction susceptibility; that is at any region when there is an earthquake or any kind of rapid dynamic load is acting, whether that particular soil can be subjected to liquefaction or not. It depends on various factors, what are those? The first one is of course, the earthquake intensity and duration. If we have higher magnitude of earthquake or higher intensity of earthquake, obviously the chances of the soil getting liquefied will be more. And also the duration of earthquake if it is more, then obviously chances are also more for liquefaction.

Then, second one is soil type. What type of soil we have? If it is a loose soil, then it is more susceptible for liquefaction than a dense material, a dense soil. Then soil relative density. In soil type, we also can say whether it is a sandy (( )) cohesionless soil or it is a c phi soil; that is if fines are present it is helpful for us. Because then we are getting some strength from the cohesion component also. So, that is why soil type plays an important factor for liquefaction analysis. Then soil relative density; as I have mentioned just now whether the sand, within sand whether it is loose sand or medium sand or dense sand depending on its relative density, the liquefaction susceptibility changes. For loose sand it will be more susceptible to liquefaction, for dense sand it will be less susceptible to liquefaction.

Particle size distribution; if the particle size distribution is giving is uniformly graded or poorly graded, in that case the chance of liquefaction is more than a well graded soil. So, well graded soil will have less susceptibility to liquefaction. Next one is presence or absence of plastic fine, which I have already mentioned within the soil type. If plastic fines are present then it will help us to to some extent reduce the liquefaction effect. So, chances of liquefaction susceptibility will be less for high plastic fines up to a certain level, it is not infinite; up to a certain level we can get this effect. So, presence or absence of plastic fines by changing the amount of plastic fines the susceptible to liquefaction can also change.

The next one is groundwater table location or degree of saturation. If groundwater table at any particular area is pretty high means very close to the ground, in that case you can expect it to be more susceptible to liquefaction than another area where groundwater table is much below the ground. Then hydraulic conductivity of the soil is also another important parameter or important factor, which influence liquefaction. How? Suppose, if the soil is having a very good hydraulic conductivity or which is another terminology is used permeability. So, if the soil is having good permeability or hydraulic conductivity very high value, what does it mean generally the sandy material.

So, in sand if we get a high value of the permeability, it means it can release the generated pour water pressure during the shaking process, earthquake shaking process easily. So, having a high value of permeability means less susceptible to liquefaction. Whereas, low permeability means it is more susceptible; because it is not getting sufficient scope to release its water.

The next one is the placement condition or depositional environment. That is how we place the soil? Whether we have compacted it or not? Whether it is loosely filled up or not? So, based on those aspects thus liquefaction susceptibility will change. That means, if we have a compacted soil obviously it will have less susceptible to liquefaction whereas, if it is a loose or filled up material, then it will be more susceptible liquefaction. Coming to next factor is aging and cementation. If the soil geologic age is very high or cementation of the soil is bonding is there, then it will be less susceptible to liquefaction. Because strength of the soil is of course, pretty high compared to less geologically young soil and on on cemented soil will have less strength.

Next factor is overburden pressure; if we can provide to a liquefied strata very high value of overburden pressure, what will happen? It will not get a chance to release the pore water pressure and to get a chance to be fully liquefied. So, having a high overburden pressure is a good thing for having a less susceptibility for liquefaction. Whereas, less overburden pressure means, it can create a high liquefaction susceptibility. That is why you will see most of the earthquake liquefaction occurs very close to the ground surface not at very long depth.

Next factor is structural load; so, this is also the same way it can be mentioned like overburden pressure. If structure load increases, chances of liquefaction for your soil will decrease. So, that is why liquefaction susceptibility for the same soil in a region where you have constructed say earthen dam of some particular height. And subsequently in the same soil you have constructed a very tall multistoried building. The earthen dam portion will be more susceptible to liquefaction because of its less structural load rather than where you have a multistoried building.

Another last factor important factor is historical liquefaction. If at any place in the history due to previous any earthquake maybe long long back, some liquefaction had occurred. Then it will be again further susceptible to more liquefaction. Why? Because once the liquefaction happens in history in some place, already the structure of the soil has been damaged. So, it will be more prone to the liquefaction. Now, what we should do to start the process of liquefaction or to start the design or construction of a foundation or geotechnical structure at a particular site?

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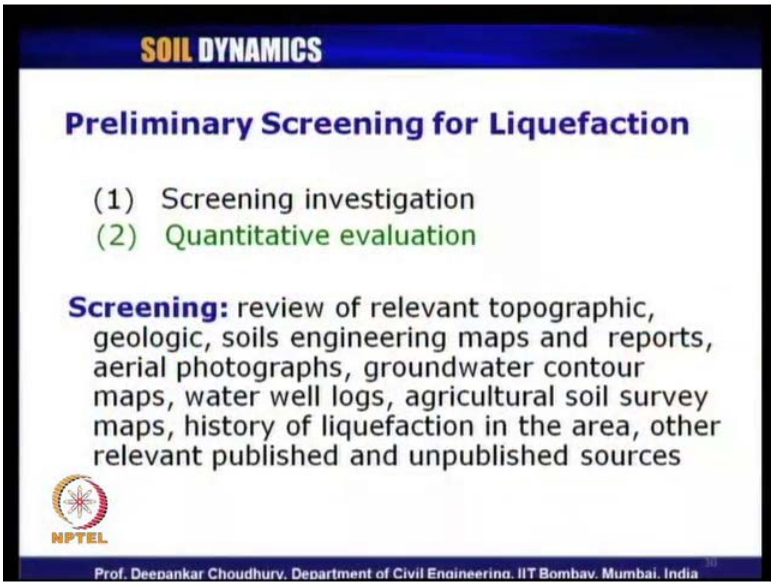

We should do the preliminary screening or feasibility study for liquefaction. As we do for the static case also, the feasibility study is very much required; that whether the soil where we are going to construct our structure, it is going to liquefy or not. So, that preliminary screening is necessary for liquefaction. Within preliminary screening, the first one is screening of investigation; screening investigation and the second one is quantitative evaluation. Means, you evaluate that how much liquefaction potential that particular soil, the site which you are going to select for your new structure or construction.

So, within the first point that is screening investigation, what we do? We generally review the relevant topography and geology of the soil; soil engineering maps and reports we collect from the history or whatever database is available. Then aerial photographs, groundwater contour map; that is how in different season in that site the groundwater changes, water well logs, agricultural soil survey map, history of earlier liquefaction in that area and other relevant published or unpublished sources. Whatever information we can get about that particular site we try to collect all those things. So, these all aspects will help us to screen that site against liquefaction study, if we want to carry out in the second stage or second point. So, we have just now learnt what are the important factors and changing of those factors, how it can influence liquefaction susceptibility; those experience we can utilize when we are handling this part of screening investigation for a particular site.

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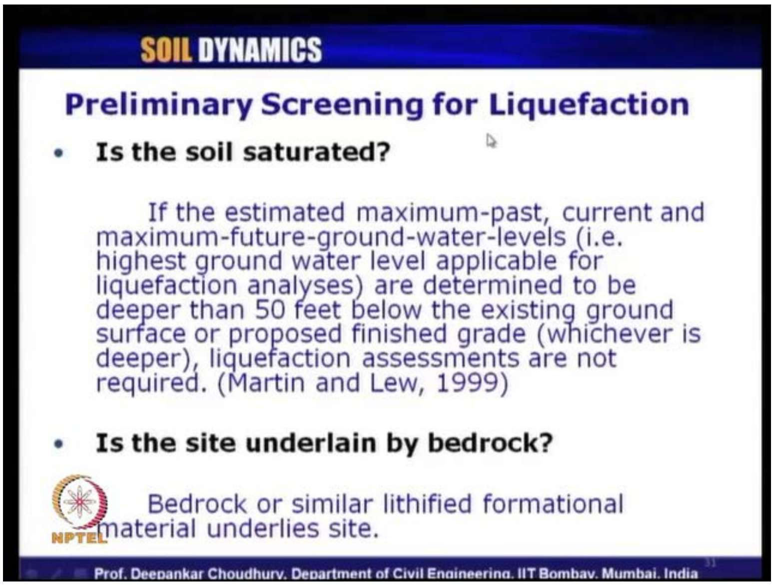

So, what we do in this? We ask our self like, is the soil is saturated in that site? If the maximum, if the estimated maximum-past current and maximum-future-ground-water levels; that is highest groundwater level applicable for liquefaction analysis are determined to be deeper than 50 feet or approximately about 15 meter below the existing ground surface or proposed finished grade whichever is deeper, whichever is deeper means? Below the proposed finished grade, suppose your foundation say pile or any other raft footing, anything is ending at a depth of say 6 meter below the ground surface or 10 meters below the ground surface. So, from that point onwards another 15 meter you go below; so within that if there is no estimated changes from the water table contour map, ground water table contour map. Then what we can suggest that liquefaction assessments are not required. That is proposed by Martin and Lew in 1999; this is worldwide used.

So, at a particular site if you get this kind of situation, then blindly you can accept that site as a good site against the failure of due to this liquefaction. Otherwise, you have to go for liquefaction evaluation; the second step. The next point is we can ask our self, is the site underlain by a bedrock? Like bedrock or similar lithified formational material underlies the site. It helps to have less susceptible to liquefaction, because bedrock obviously rocky material cannot liquefy. We have seen the condition; cohesionless soil material only liquefies. So, if we have a underlain bedrock, chances of liquefactions will be very minimum. So, that will be again another good site for selection for your new construction.

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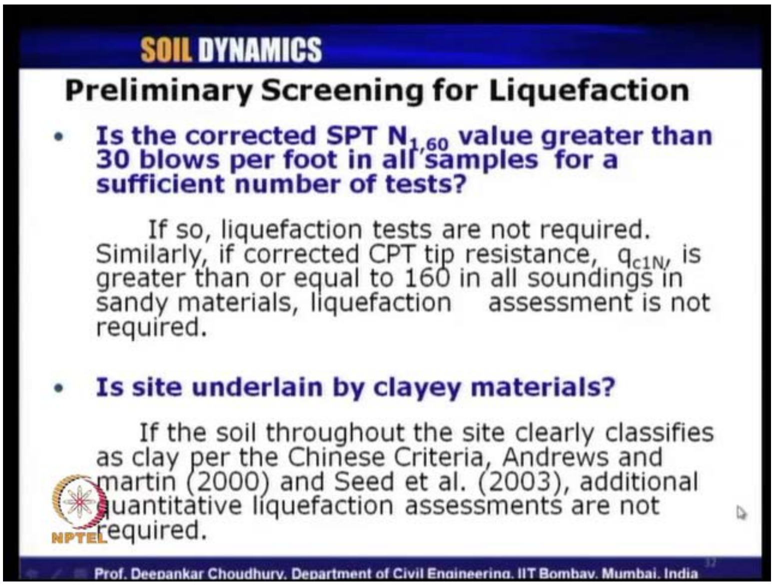

Next question we can ask our self, is the corrected SPT N1 60 value, standard penetration test corrected value, N 1 60; is it greater than 30 blows per feet in all the samples for a sufficient number of test? That is in the particular the site say you have conducted n numbers of SPT test. And the observed value you need to correct it for several correction factors which we will discuss soon. You know that what are the SPT correction factors? So, after correcting all those, the corrected value if you get as N 1 60; if this corrected value is greater than 30, then what inference we can draw? If it is so in that case, liquefaction test are not required or liquefaction evaluation are not required. We can recommend that it is a good site.

Similarly, if the corrected cone penetration test, CPT tip resistance value q c 1 N, q c's raw data which you record from c p t test and q c 1 N is corrected CPT value after doing all the corrections, which we will see soon in the next lecture. Is greater than or equal to

160 in all soundings in sandy materials, then also liquefaction assessment is not required. And this 160 is in FPS unit; remember this is in FPS unit. And another question we can ask is site underlain by clayey material? If the soil throughout the site clearly classifies as per the Chinese criteria, Andrews and martin criteria and seed et al criteria. Then additional quantitative liquefaction assessments are not required; if it is underlain by clayey material, if it follows this Chinese criteria or seed et al 2003 criteria. So, these are the basic things we need to see, while doing the preliminary screening of a site subjected to liquefaction or not; suitability of a site.

Now, let us come to the quantitative assessment of the liquefaction; that is second stage of or more correct way technically to compute the liquefaction potential of a site for selection of the site for new construction or something like that.

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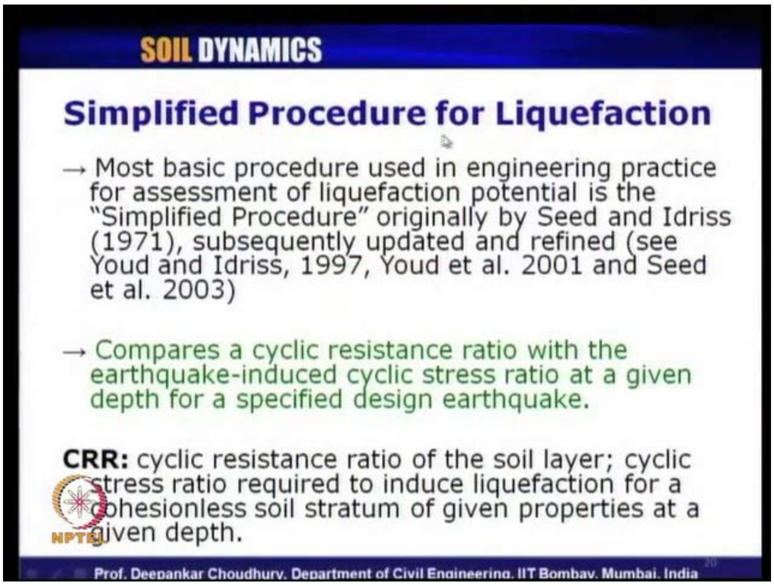

So, simplified procedure for computation of liquefaction; most basic procedure is this simplified procedure. It is used in all engineering practices for assessment of liquefaction potential. What is liquefaction potential? That is, how much your soil is going to the susceptible for liquefaction. That is how much potential it has to get liquefied when an earthquake or a rapid dynamic loading is acting on the soil.

So, liquefaction is the simplified procedure and that simplified procedure, this nomenclature was originally proposed by Seed and Idriss in 1971. So, that was the pioneering work or the first work of the available in the literature to compute a liquefaction potential of any soil using the simplified procedure as was proposed by Seed and Idriss in 1971. In this connection, I can mention professor Idriss is currently an emeritus professor at university of California Davis. This was his Ph.D work, he was the student and he was the professor under whom he did the Ph.D; Professor Harry Bolton Seed.

H B Seed, he was the professor at university of California Berkeley; he has expired long back. Currently, his son his name is Ray Seed, he is also a professor at university of California Berkeley. So, this Seed refers to older Seed, they called old Seed. So, Seed and Idriss in 1971, they proposed this basic method of computation of liquefaction potential. And subsequently it has been updated by several researchers and it has been refined. You can see these are the references like Youd and Idriss; Youd did again his work under the supervision of professor Idriss, in 1997. Youd and Idriss all these are ASCE paper; that is American society of civil engineers journal paper.

Then, Youd et al 2001 and seed et al 2003, this third paper Seed is the younger Seed; that is the son of the older seed, this is Professor Ray Seed. Now, he also must be about age of 50 years or so and professor Idriss is now about 75 years or so. So, in among all these papers this is considered as a kind of state of the art kind of paper, Youd et al 2001; so, you please note down this paper. In 2001, in journal of geotechnical and geo-environmental engineering of ASCE, American society of civil engineers, this paper was published Youd et al. This was co-authored by several numbers of researchers about 20 co-authors or so.

It is not a kind of original paper, it is a kind of assembling all the research outputs done till date; that is till 2001 by several researchers in the field of liquefaction. So, that is why as I have mentioned it is state of art kind of paper which gives till date, the final or the actual guideline how to compute the liquefaction potential of any type of soil, using the SPT test results or CPT test results or SASW test results. So, in this paper you will find several other researchers work has been incorporated and mentioned. And this paper was the outcome of an workshop; they have conducted at California one workshop to discuss about the standardization of the method to compute the liquefaction potential of soil.

So, that is why all the (( )) those who did research in the area of computation of liquefaction, they all sat together and they decided, ok these are the final proposition till date which can be followed by any designers or practitioners worldwide irrespective of place to compute the liquefaction potential of any soil. So, that is why worldwide this method is commonly used nowadays, Youd et al 2001. Earlier everybody used to use this Seed and Idriss 1971 approach. So, that has been subsequently has been modified and updated in this Youd et al 2001. So, I will ask you to refer to this paper, to go through this paper thoroughly by each and every line. Then you will get clear outcome of what are the design steps has to be used. I will also mention in my lecture here.

So, what simplified procedure means? For computation of liquefaction potential, it compares a cyclic resistance ratio. They have defined a new terminology; cyclic resistance ratio. It is also called CRR, you can see the first word of this. So, CRR means Cyclic Resistance Ratio with the earthquake-induced Cyclic Stress Ratio at a given depth of specified designed earthquake. So, the second one is CSR; so, CRR and CSR are compared for a particular earthquake at a given depth and their ratio will give the factor of safety against liquefaction.

Now, what is CRR? Cyclic Resistance Ratio of the soil layer; this is the Cyclic Stress Ratio which is required to induce the liquefaction for a cohesionless soil stratum of given properties at a given depth. So, this Cyclic Resistance Ratio is nothing but it is the amount of Cyclic Stress Ratio which is required to liquefy your soil at a particular depth and for a given earthquake, fine. So, earlier in this connection, let me mention to you earlier in the simplified procedure, if you look at the original paper of seed and Idriss 1971 which you should also read because to know the development of the method, they had used the word Cyclic Stress Ratio here also. Later on Youd et al 2001, they have modified the terminology to Cyclic Resistance Ratio. I will show you through equation so it will be more clear to you.

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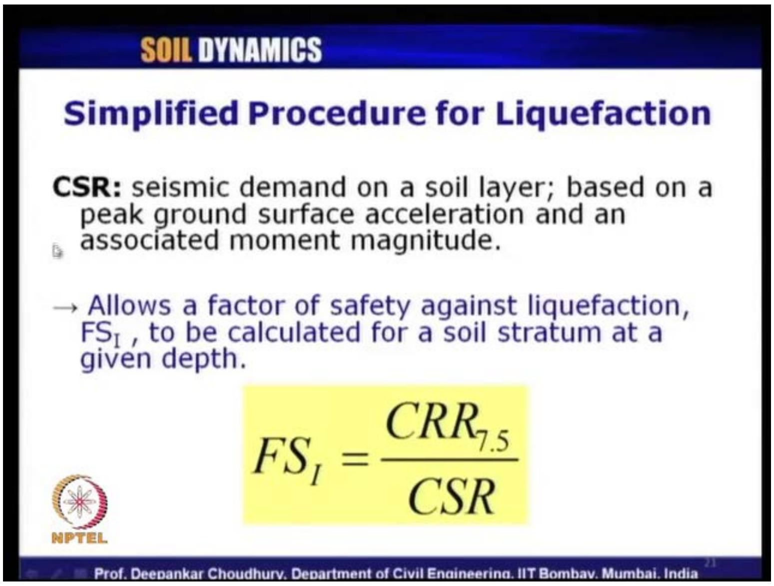

What is CSR, Cyclic Stress Ratio? This is nothing but the seismic demand of a soil layer; based on a peak ground surface acceleration and it is associated with a moment magnitude. So, finally how the liquefaction potential is calculated? It calculated in terms of a factor of safety. So, it allows a factor of safety against liquefaction; say FS I to be calculated for a soil stratum at a given depth. Remember, this point, this factor of safety against liquefaction is calculated at a given depth. So, with respect to depth this will keep on changing.

So, at one particular depth we will get one particular value of the factor of safety against liquefaction. And how that factor of safety against liquefaction is computed mathematically? This is the expression CRR, Cyclic Resistance Ratio suffix 7.5 divided by CSR; that is Cyclic Stress Ratio. What is this 7.5 indicates? This 7.5 means, the CRR that is Cyclic Resistance Ratio of the soil is calculated for earthquake of a moment magnitude of 7.5. So, earthquake they have several scale.

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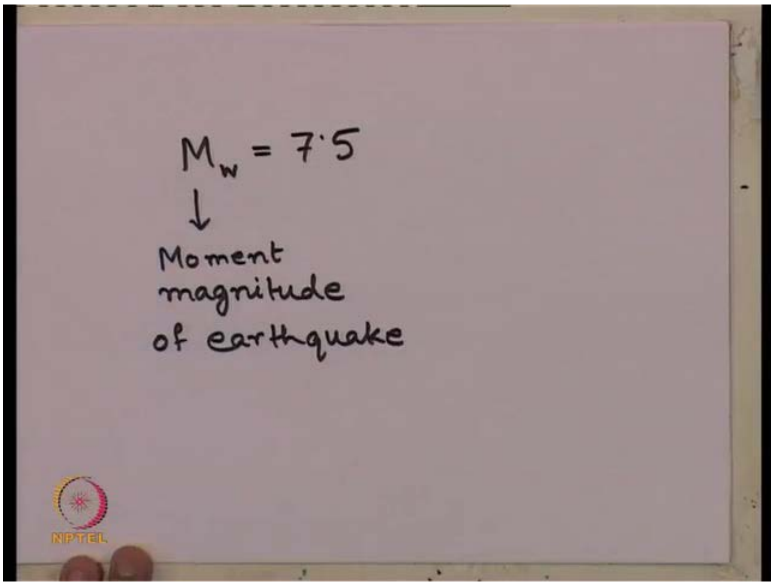

What I am telling here? It is for M w of 7.5; M w is called moment magnitude of earthquake. There are several earthquake magnitude determination scale; among them, this is technically most correct. Technically, for all computations etcetera, engineers use the value of this M w; M suffix w, it is called moment magnitude of earthquake. When the value of moment magnitude of earthquake is 7.5, this CRR is computed at that value. Because the soil it is resistance power with respect to a particular earthquake needs to be standardized. Otherwise, at different level of earthquake it is resistance will be different, right. So, that is what it is applied here; the value of CRR suffix 7.5 to standardize at what value of earthquake magnitude the computation has to be done. Now, if your earthquake is any other value other than 7.5 which is quite common and obvious, we need to do a correction to our value to bring it to the reference value of CRR 7.5.

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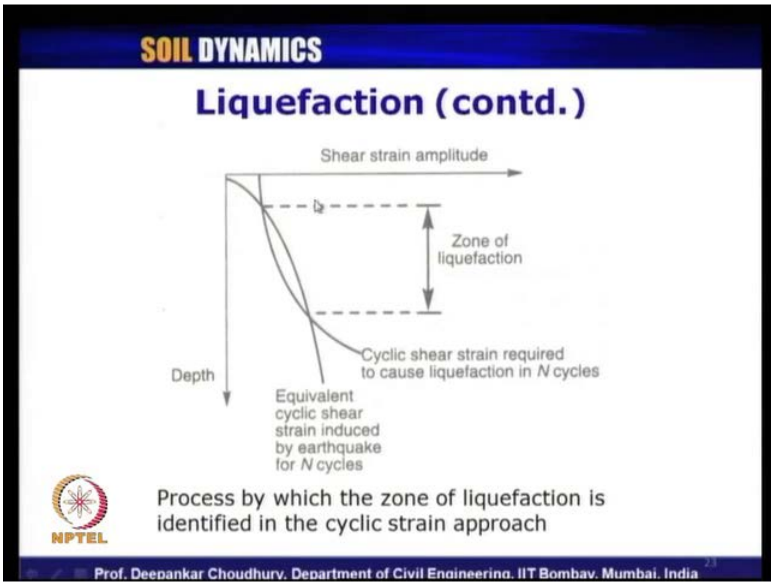

So, what we are explaining? By figure we can mention, that is process by which zone of liquefaction is identified; it is like this. The development of the shear stress versus depth has been plotted here. This curve, this gives the equivalent cyclic shear stress induced by an earthquake. What does it mean? This is nothing but our CSR; this is giving us the value of CSR; that is the denominator of the previous equation of factor of safety. Whereas, this graph is giving us cyclic shear stress required to cause liquefaction. So, this is the capacity of the soil, the resistance power. That is how much cyclic shear stress maximum is required to cause the liquefaction. And this one is giving what is the cyclic shear stress is actually getting induced for a particular earthquake.

So, what does it mean? This is your capacity and this is how much actually is occurring; so if it exceeds this value, here is our zone of liquefaction. Because in this case, the cyclic shear stress which is produced in the soil during an earthquake is more than the resistance power of the soil or the capacity of the soil, to take the cyclic resistance or cyclic shear stress. So, that is why you can clearly see this zone where this CSR exceeds the value of CRR is the zone of liquefaction. In other words, if the factor of safety is less than 1 in the previous slide, if this factor of safety is less than 1; obviously that means the soil has liquefied at that depth. And if it is greater than 1 obviously it is safe against liquefaction. So, this is in terms of cyclic shear stress; let us see what happens in terms of cyclic shear strain.

So, similarly, we can say that process by which zone of liquefaction is identified using cyclic strain approach; this is shear strain amplitude versus depth. This one gives the actual equivalent cyclic shear strain which is induced by an earthquake for n numbers of cycles of earthquake. And this line or this curve give us the distribution of the capacity of the soil; that is how much it can sustain or withstand the cyclic shear strain till the liquefaction occurs in n numbers of cycle. So, obviously again here we can find out this zone is zone of liquefaction; where the equivalent cyclic shear strain developed in the soil is increasing your how much is its capacity.

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Now, as a designer what value of factor of safety we should use? For all geotechnical design problems, we know though the factor of safety one is showing us the condition of just limiting condition of equilibrium; that is the limiting stage of failure. But for design we do not use the factor of safety equals to 1; we use little higher value. So, depending on the type of problem, depending on uncertainty and several other factors we decide on what are the factor of safety we should use for design purpose. Similarly, for computation of liquefaction potential of a soil the acceptable factor of safety is recommended as 1.3.

So, factor of safety for design, if you are doing the design of any geotechnical structure at a earthquake prone region, check that all the soil layers at various depth whichever you should consider as a designer must have a factor of safety against liquefaction as 1.3

if the structure which you are going to construct or the foundation which you are going to construct in that zone is very important structure. Otherwise, if it is not that important structure of factor of safety against liquefaction of 1.1 is good enough. So, it depends on the importance of your foundation or underground structure or substructure which you are designing for.

So, that is what it is mentioned here 1.3 is recommended, but depends on severity of the hazard. That is if, suppose that zone is very much in the high magnitude of earthquake seismic zone importance of the structure and vulnerability of the structure. Vulnerability means, suppose that structure involves a hospital building; for example. It is more vulnerable structure; if any disaster occurs. Because there all the patients that is the place where we keep all the people after taking them out from a disaster prone region. So, if that hospital building itself is a situating in the disaster prone region, we have to check it is vulnerability etcetera. And depending on that a higher value of factor of safety should be used for design.

Then the tolerable settlements of the building or the foundation or level of risk, how much risk level is associated? Suppose, in a building where school building, where small kids are there; so obviously level of risk is much more than a conventional house, where all aged people can stay. So, that way the importance level of risk etcetera has to be seen and factor of safety 1.3 is recommended. Whereas lower factor of safety as I said 1.1 is also acceptable, for say suppose single family dwellings. For example, where potential for lateral spreading is low; lateral spreading is another phenomenon which occurs after the liquefaction.

So, once the soil gets liquefied means it is flowing as a fluid. So, what will happen? It will flow from high gradient to low gradient as the water does. So, the same way the soil, that muddy soil or muddy water or muddy fluid will also spread laterally; that is why the terminology used is lateral spreading. So, lateral spreading is the after effect of liquefaction. And we have procedure also to compute how much lateral spreading, up to what extent or up to what distance it will affect all these things are available. But I am keeping that beyond the scope of this course, because that we cover in another course of earthquake geotechnical engineering. Then the differential settlement is low and hazard related to the differential settlement, if it is low in those cases we can apply the lower factor of safety of 1.1. So, these are the two recommended values of factor of safety for any design in an earthquake prone region to check about liquefaction potential. So, with this we will complete today's lecture; we will continue our lecture in the next class.