Soil Dynamics
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Module - 4
Dynamic Soil Properties
Lecture - 24
Becker Penetrometer Test (BPT), Cone Penetrometer Test (CPT), SPT v BPT, SASW Test

Let us begin our today's lecture on soil dynamics. We were continuing with our module 4 that is dynamic soil properties.

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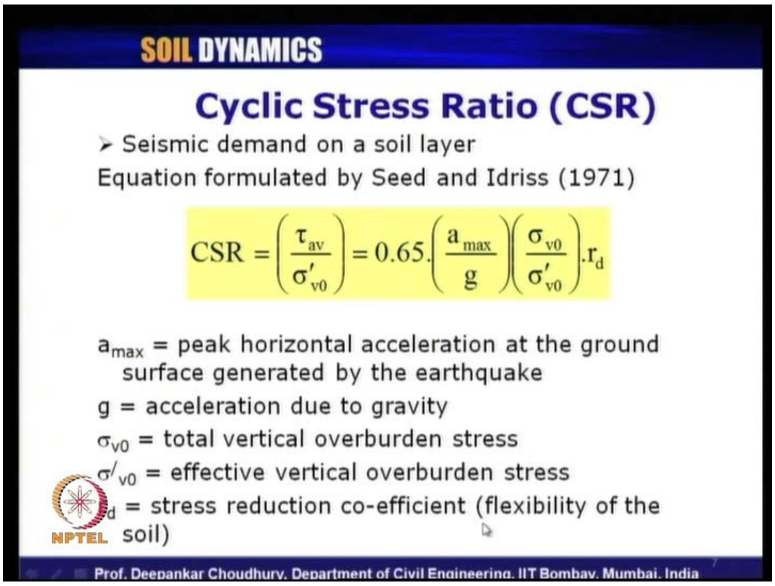

A quick recap what we had studied in the previous lecture. We have seen how to compute the cyclic stress ratio or the CSR as per the formulae given by Seed and Idriss in 1971. This is the formulae to compute the CSR. This is the non-dimensional parameter, which we are computing. From the knowledge of our peak horizontal acceleration at the ground level generated due to an earthquake, g is acceleration due to gravity, and at which particular depth we are calculating the liquefaction potential, we can find out the total vertical overburden stress divided by effective vertical overburden stress at that point; times r d is nothing but the stress reduction coefficient due to the flexibility of the soil.

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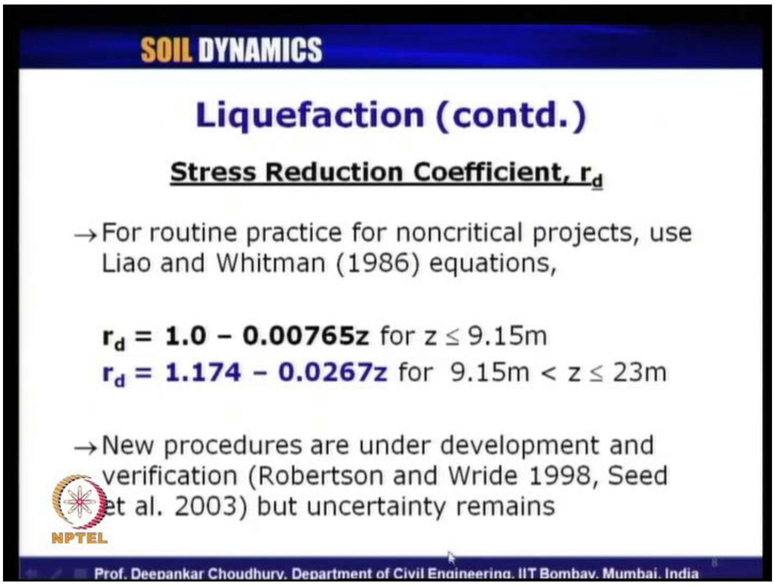

And to compute the stress reduction coefficient r d, Liao and Whitman had proposed two equations; simple linear equations for different ranges of depth below the ground surface.

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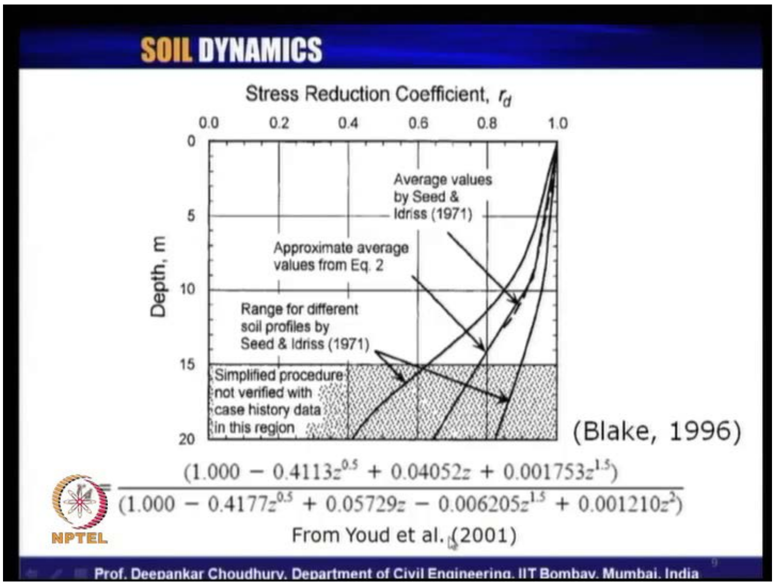

Further which has been modified by Blake in 1996 and he proposed; this is the relationship to compute r d for known value of z for any value of z and it has been recommended in the paper by Youd et al. 2001.

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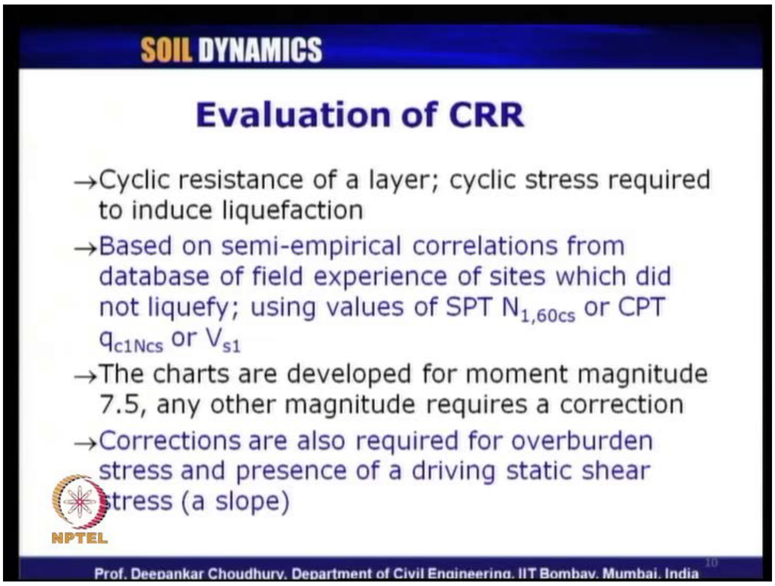

Then, we have seen what are the steps to evaluate cyclic resistance ratio or CRR. We can compute the value of CRR from the field test results of standard penetration test SPT N value or cone penetration test CPT q value or SASW test from the shear wave velocity V s value. And remember from all the corrected values of these data, we can correlate with respect to CRR and then can compute the factor of safety against liquefaction.

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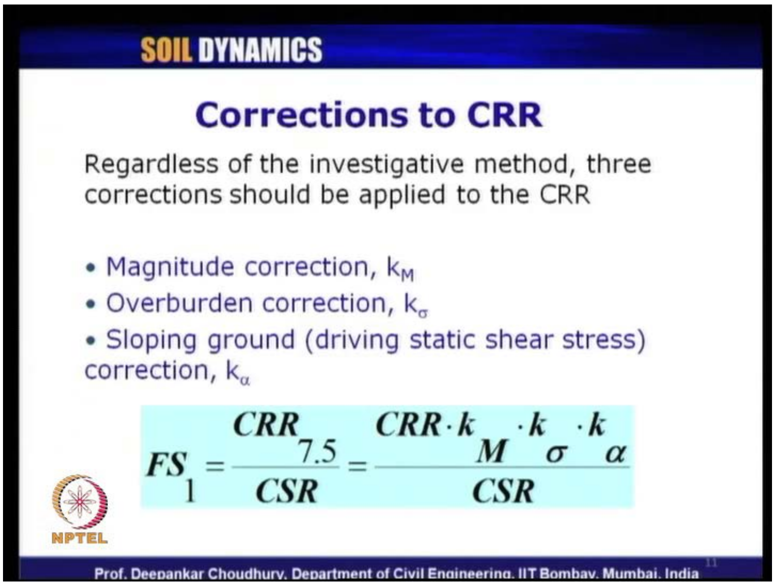

So, what are corrections involved to the value of CRR? We have seen there are three major corrections; magnitude correction factor K M, overburden correction K sigma, and sloping ground correction K alpha. So, the factor of safety against liquefaction which was defined earlier as CRR 7.5 by CSR, it can be written by applying the corrections; three corrections of two CRR like this, CRR at any magnitude of earthquake times this magnitude correction factor K M times the overburden correction factor K sigma times the sloping ground correction factor K alpha divided by the CS.

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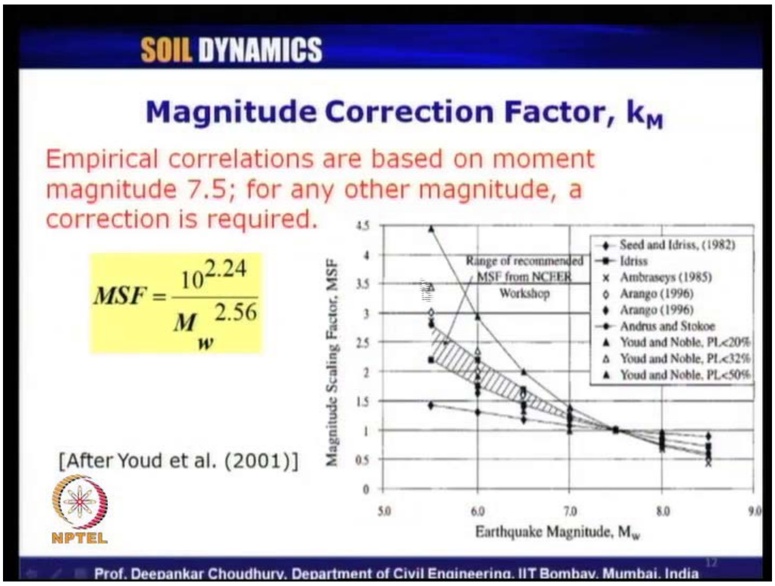

We have seen how to compute the magnitude correction factor K M. K M at moment magnitude earthquake moment magnitude of 7.5 it is one. So, no correction is required. Otherwise as moment magnitude less than 7.5, the magnitude scaling factor or magnitude correction factor will be more than one and if the earthquake magnitude is more than 7.5, then the correction factor will be less than one. And this equation has been proposed which is given in Youd et al. 2001 to compute for any moment magnitude of earthquake how much is the correction factor K M has to be used.

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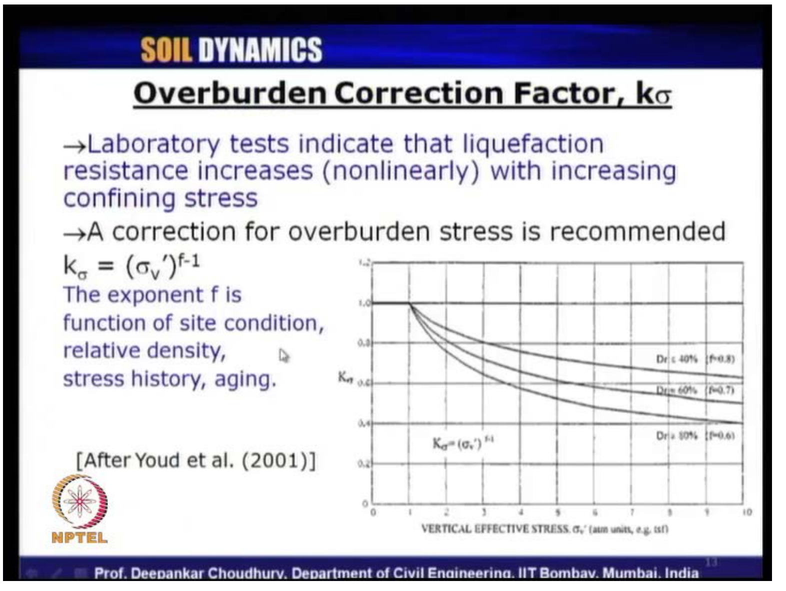

Also we have seen that how to compute the overburden correction factor; overburden correction factor K sigma is expressed by this equation where sigma v dash is the effective vertical stress at the point where we are calculating the liquefaction potential and this exponent f is a function of relative density stress history and aging of the soil. And as this chart was given, up to a effective vertical stress of one atmospheric pressure which is equivalent to about 100 kPa; no correction is required. But beyond that correction factor is less than one depending on the amount of relative density of the cohesion less soil. If it is less than equals to 40 percent, then the exponent factor f is 0.8. If it is 60 percent it is 0.7; if it is greater than or equals to 80 percent, then it is 0.6 in this equation.

And remember when you are using this equation, as this is an empirical equation, we have to be very careful about the units. So, sigma v dash unit has to be in atmospheric unit or tsf unit; do not use kPa unit while using this equation, otherwise you will get absurd value of K sigma. So, whenever you are doing the calculations in SI unit to sigma v dash effective vertical stress, convert it to equivalent atmospheric pressure and put that value of atmospheric pressure in this equation to get a value of K sigma. Otherwise you use this design chart; whatever you feel better, both the thing will give you the same value of K sigma.

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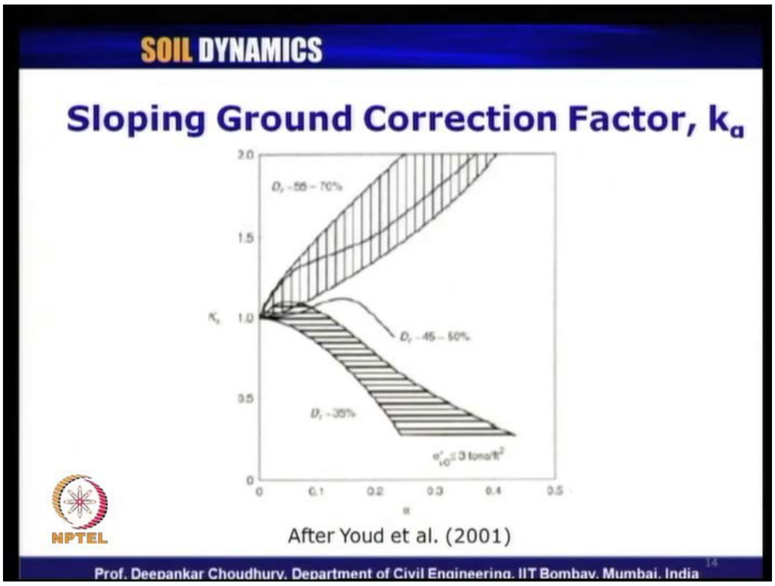

Then sloping ground correction factor K alpha, we have seen if the ground is horizontal no correction factor is required; then correction is one. Otherwise depending on different angle of the sloping ground and also depending on different relative density of the soil, if the effective vertical stress is less than 3 ton per square feet; in that case we will get different values of K alpha.

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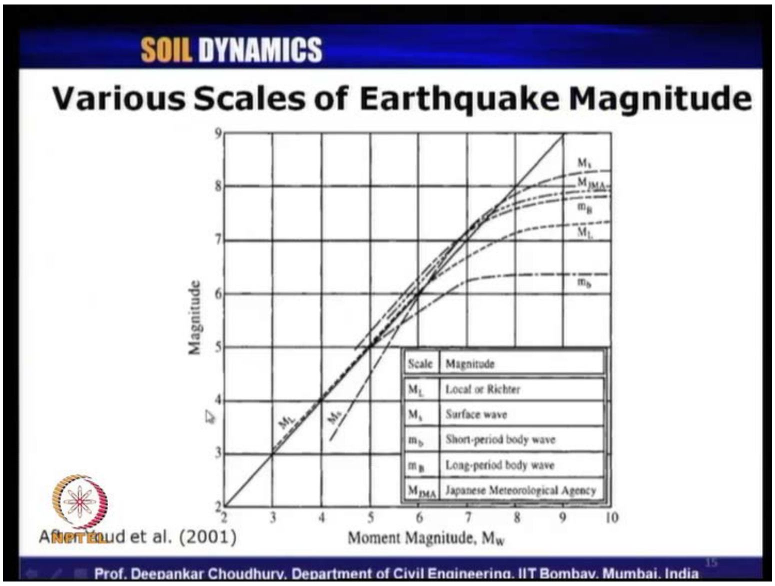

Then we had also seen how to correlate different scales of earthquake magnitude with the moment magnitude scale, because finally all our calculation has to be done in terms of earthquake moment magnitude. So, if the given input data is for any other earthquake scale. We should use this conversion chart or conversion table to find it out in terms of moment magnitude of earthquake.

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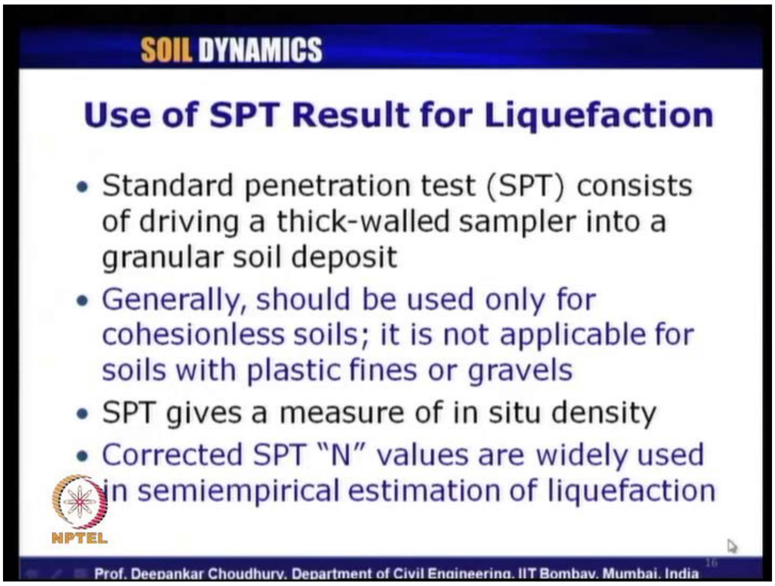

Now to use the SPT that is standard penetration test result for liquefaction calculation, we have seen what are the steps, we have to use the corrected N value.

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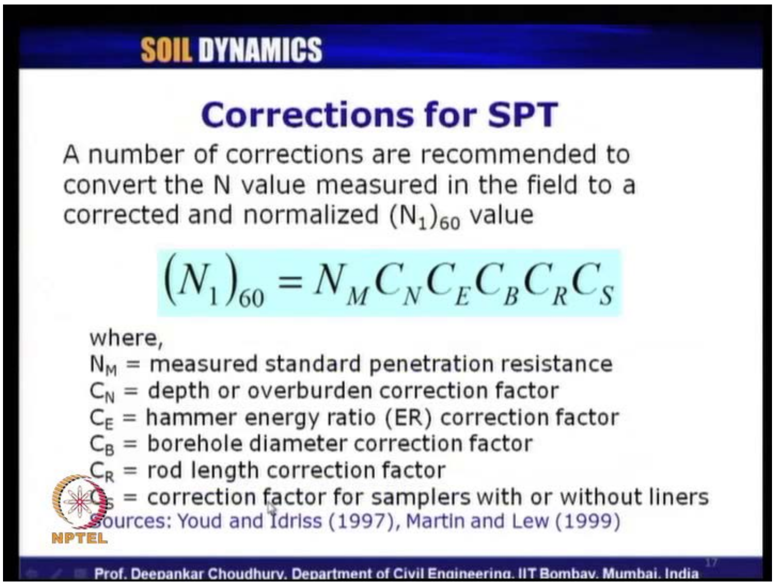

And the corrected N value is computed by using this expression where N M is the raw or measured value of standard penetration number at the site, from the last 30 centimeter penetration the number of blows required for that in the standard SPT test. And what are the different correction factors we have seen? C N is the overburden correction, C E is energy correction, C B is borehole diameter correction, C R is rod length correction, and C S is sampler correction factors.

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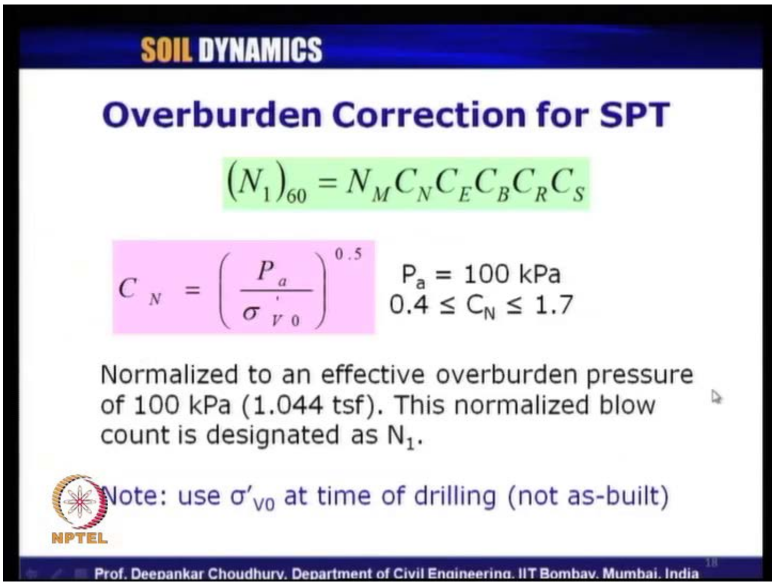

Then how to compute the overburden correction C N? This is the expression and the value of C N lower limit or upper limit is also given. It is again standardized with respect to one atmospheric pressure or 100 kPa.

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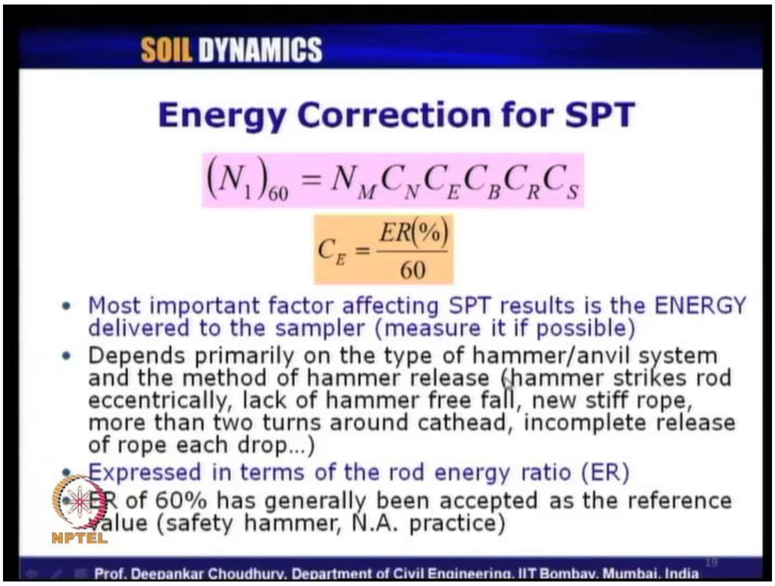

Then energy correction factor is computed by how much energy is transferred actually to the sampler tube by your particular hammer divided by 60; that is 60 percent energy is considered as the reference.

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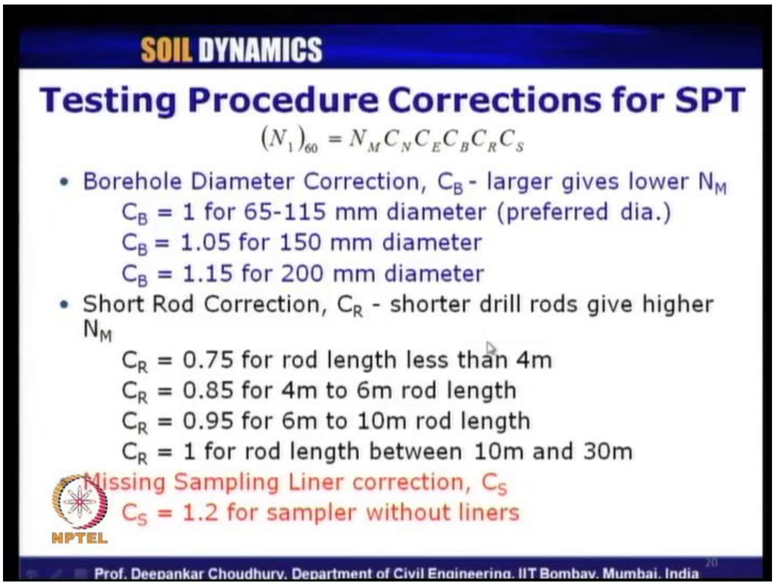

Then testing procedure corrections like borehole diameter correction we have seen no correction is required if the borehole diameter is between 65 to 115 millimetre diameter; these are preferred diameter. Then short rod correction factor also we have seen if the rod length is about 10 meter to 30 meter, then no correction is required; otherwise correction factors has to be adopted. And the sampler lining correction factor C S if there is liner present in the sampler tube no correction is required, but without liner we have to use the correction factor.

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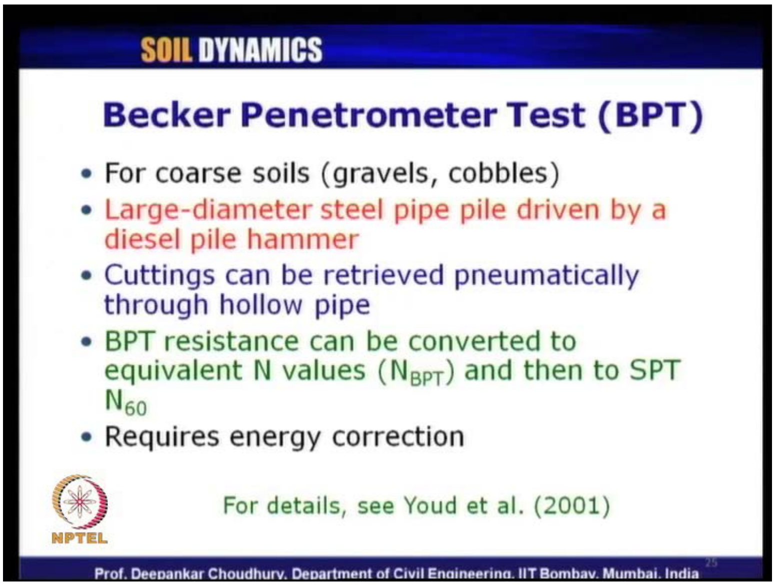

So, what is BPT? BPT is applied for coarse soils that is gravels even larger than gravels, cobbles. So for them, we generally use Becker penetrometer test. Here what is the basic difference from the SPT? Here large diameter steel pipe pile driven by a diesel pile hammer is used instead of a small diameter sampler tube and cuttings can be retrieved pneumatically through the hallow pipe. BPT resistance can be converted to equivalent N values; that is N BPT it is called; that is here also we count the number of blows required to penetrate a certain distance. So, that is called N value obtained from BPT test and then it can be correlated to the SPT N 60 value which is further can be used for liquefaction analysis. Here also energy correction is required very much. So, for details you can see Youd et al. 2001 paper once again.

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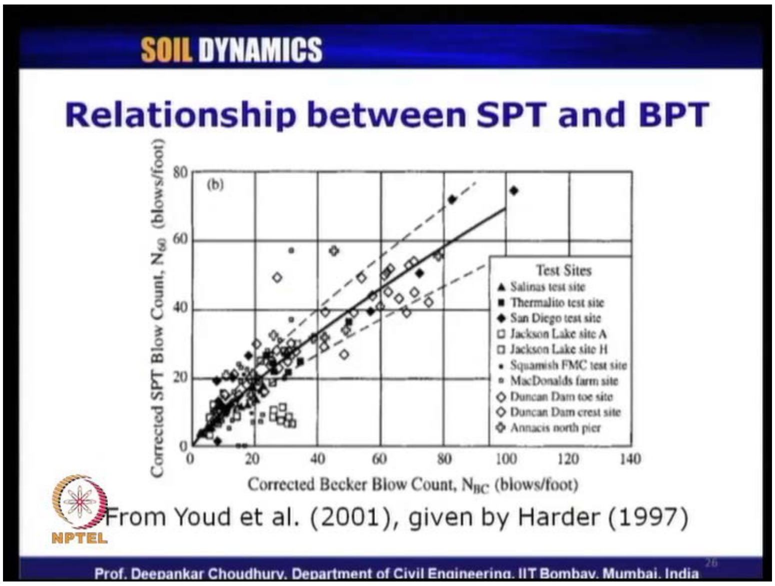

So, this is the relationship between SPT and BPT which is given originally by Harder in 1997 which is mentioned in Youd et al. 2001 also. You can see the correlation; this y-axis is corrected SPT blow count N 60 and it is measured in blows per foot and this x-axis gives the value of corrected Becker blow count from BPT test N BC in terms of blows per foot. So, if you have used BPT test or Becker penetrometer test at a site, you will get N BC value; use this chart, get a corresponding N 60 value and then use that N 60 value for your liquefaction analysis. So, that is the procedure mentioned.

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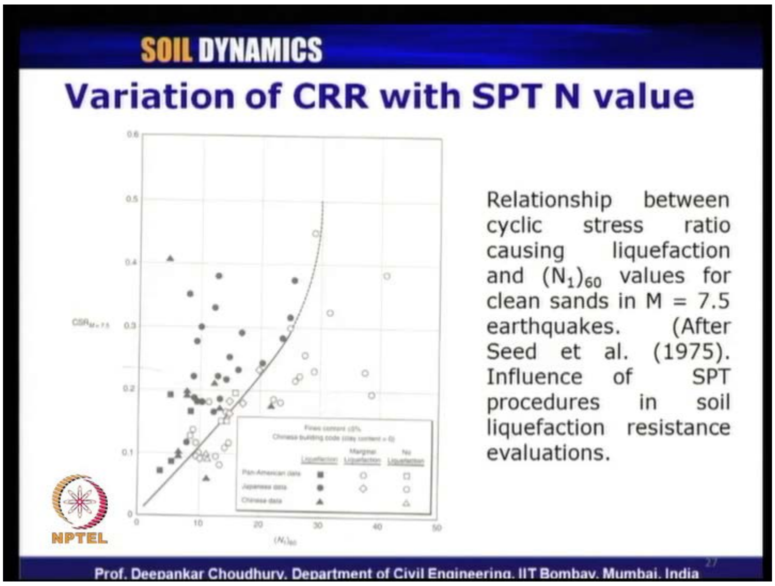

Now, let us see how this CRR where is with SPT N value? Look at this chart; it was given by Seed et al. 1975. This is done by Professor Harry Seed, the older Seed; 1975 this chart was produced. So, y-axis is given as CSR with suffix M equals to 7.5. As I have mentioned in the previous lecture that earlier Seed and Idriss have mentioned this CRR also as a cyclic stress ratio for moment magnitude of 7.5 divided by cyclic stress ratio which is getting generated. So, capacity by whatever is getting generated. So, that is why they used CSR.

But remember as per the latest notation is concerned, as per Youd et al. 2001, this is nothing but our CRR cyclic resistance ratio. So, that is why I tried to give you the older chart first which was given by Seed et al. in 1975, then I will give you the latest chart where this has been written as CRR. Both are same thing; only the name has been changed or updated. So, what they have done; this graph what it shows? It shows relationship between cyclic stress ratio causing liquefaction and (N 1) 60 corrected values for clean sand.

So, again this is (N 1) 60 CS actually in M W of 7.5 earthquake. So, influence of SPT procedures in soil liquefaction resistance evaluation has been obtained here. What this graph shows? Look at here this legend; fines content is less than 5 percent. So, this line indicates for the soil having fines content less than 5 percent in it. And Pan American data, they have collected Japanese data they have collected, and Chinese data they have collected; for three categories they have divided where from previous history of earthquake, they have gathered this field data or field information due to earthquake in earlier history; that is before 1975 which are the places the liquefaction occurred where no liquefaction occurred and where marginal liquefaction occurred.

So, these are the symbols. This shows from the Pan American data, these are Japanese soil data, and these are Chinese data. They have collected across the world mostly from these three regions; then they have plotted it after calculating that CRR or the CSR M 7.5 versus the recorded value of (N 1) 60 corrected to clean sand condition. And this line what it shows? It shows a kind of boundary line between the two cases where liquefaction occurred and not occurred. So below the line, this is the zone mostly where liquefaction did not occur; though they are two points you can see this point and this point, but mostly, they say this is the boundary line to relate to correlate between this CRR and (N 1) 60 so that between a non-liquefied zone and a liquefied zone. So, this relationship is clearly obtained from the collected field data from historical earthquake till 1975.

But you know that after 1975 also several numbers of very major strong earthquakes have occurred at several parts of the world where liquefaction also has been measured, means triggered liquefaction has been seen and also (N 1) 60 values were available. Later on liquefaction analyses also have been carried out; so, so many other field data are available after 1975. So that is why, this is an area of continuous research which needs updation from one earthquake to another earthquake; that is why most of the parts of geotechnical earthquake engineering is very recent in nature or very vibrant or very modern in nature, because once an earthquake major earthquake occurs from the field investigations of this phenomenon like liquefaction, one can get more data point which will give the community researchers community to identify what is actual boundary between a liquefied zone and a non-liquefied zone in terms of this CRR and measured SPT value corrected SPT value.

So, that is why even this chart design chart proposed by Seed et al. 1975 has now become almost obsolete. Even his son Professor Ray Seed also does not believe on this results and he said it needs to be updated. These are almost wrong because many of the new data, new observations, new things have been found. So, it needs continuous updation; that is what I want to project to you that this is not a steel topic or already knowledge has been gathered that kind of topic. It is a continuously updated topic where updation is required from one earthquake observations to the another earthquake observations, because it is based on the collection of the field data.

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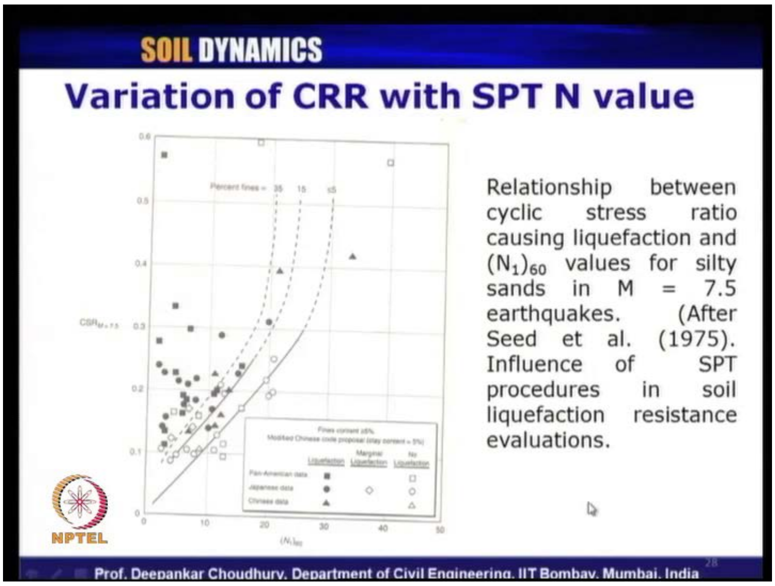

Similarly, some more design charts were given by Seed et al. in 1975 for different parts and fines. The previous curves was for parts and fine less than 5 percent that line. Now they have given for 15 percent and 35 percent fines also where those lines are coming for different CSR versus (N 1) 60 plot in the same graph. So, you can identify some more liquefied zones are coming when you have more percent fines value in the corrected computation of (N 1) 60 in terms of clean sand.

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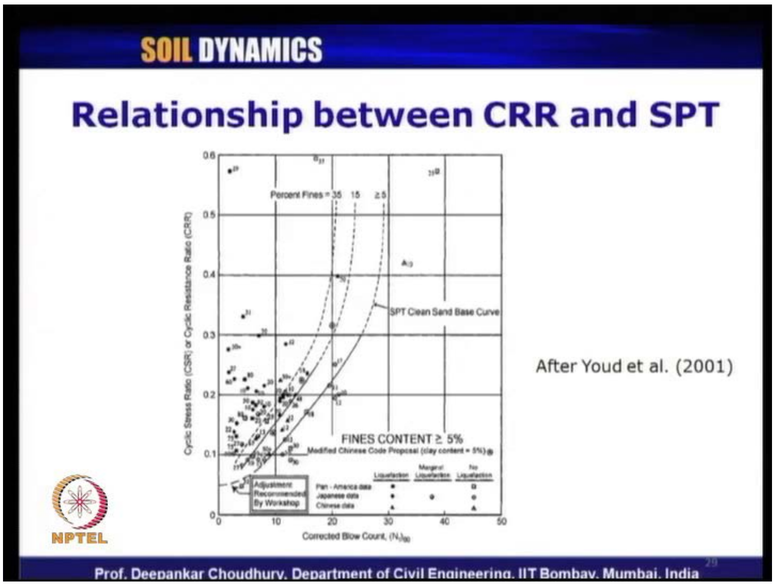

And this is the latest curve which is given in Youd et al. 2001 which is used worldwide to compute CRR from SPT (N 1) 60 value, corrected (N 1) 60 SPT value in terms of clean sand condition. Based on percent fine greater than equals to 5 percent, this line has to be used; for 15 percent this line has to be used, and for 35 percent this line has to be used. So, here several more data have been collected and put in this chart. So, that is why in the y-axis they have mentioned this is cyclic stress ratio at 7.5 earthquake or the new terminology cyclic resistance ratio.

Now instead of using this chart in the design, what you can do; from your calculated (N 1) 60 corrected value, you go to this chart, depending on your percent fine of the soil. Suppose your corrected (N 1) 60 value is 20, you go up to this; if it is percent fine is 5 percent go to this graph, project it here. Your CRR value is something about say 0.22. So, in this way from this design chart you can read the value of CRR; that is the procedure to compute the value of CRR from your measured and corrected SPT N value. So, in this way from this design chart you can read the value of CRR; that is the procedure to compute the value of CRR from your measured and corrected SPT N value.

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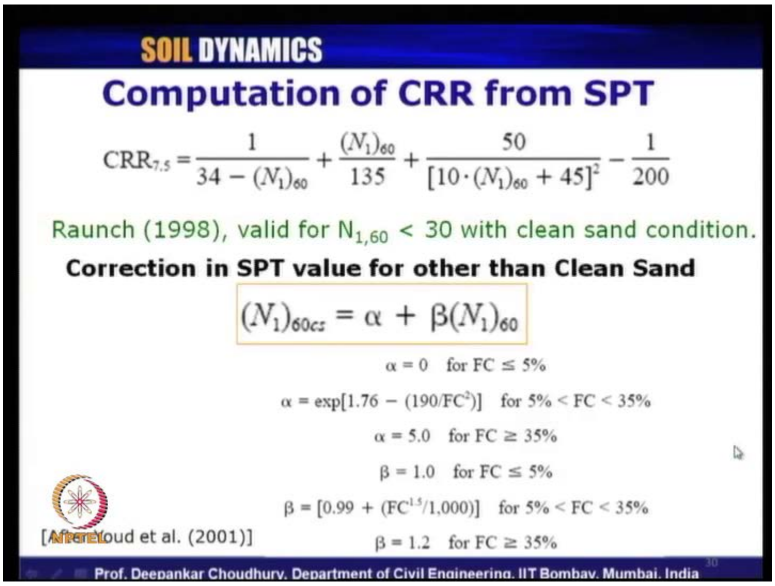

Or you can use empirical relationship; this equation which was given by Raunch in 1998 and this equation is valid for the corrected N 1, 60 after applying the clean sand correction, if the value is less than 30 then only this equation is valid. So, CRR 7.5 value you can directly get by using your (N 1) 60, then clean sand corrected value using in this equation provided it is less than 30; you can compute the value of CRR. So, basically there are two methods; one either you can use the design chart or you can use this equation proposed by Raunch 1998. Both will give you almost same value of CRR and for fines correction what I said earlier, the same expression in a different format is given (N 1) 60 CS can also be obtained; that is the corrected SPT value in terms clean sand can also be obtained using this equation alpha plus beta times (N 1) 60.

If fines content is less than 5 percent, in that case, alpha is 0 and beta is 1; that means, no clean sand correction is required. But if the fines content is between 5 to 35 percent then alpha is computed by these expression; that is e to the power 1.76 minus 190 by FC square, F C is in percent value and for fines content between 5 percent and 35 percent the factor beta is calculated by using this equation 0.99 plus FC fines content in percent to the power by 1.5 by 1000. If the fines content is greater than 35 percent, in that case it is proposed alpha is 5 and beta is 1.2. So, this is actually a better equation than the previous one to compute the corrected (N 1) 60 clean sand value because for all ranges of fines content below 5 percent, 5 to 35 percent, and greater than 35 percent are given here.

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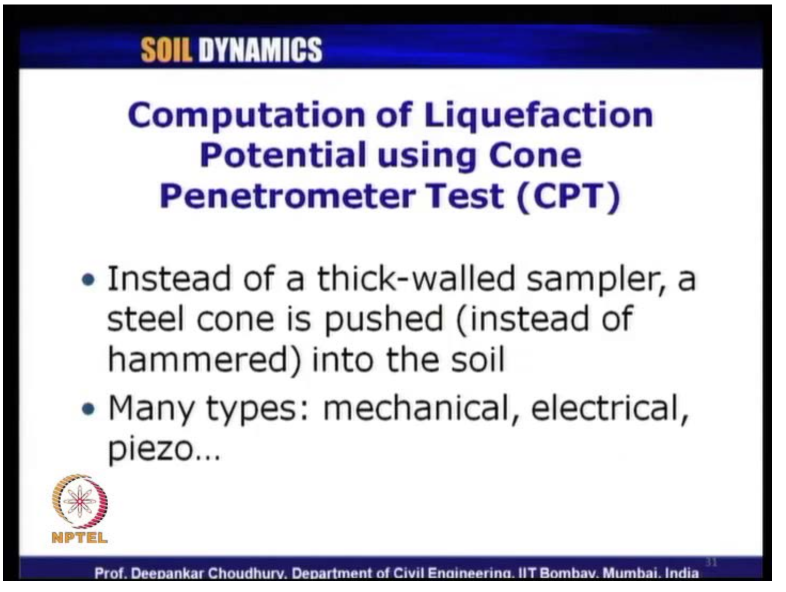

Now using SPT standard penetration test number, how to calculate the liquefaction potential or the factor of safety against liquefaction potential we have seen because CRR we have computed, CSR earlier we have computed; their ratio will gives us the factor of safety against liquefaction. If we want to use the other test; that is cone penetration test or cone penetrometer test CPT, how to make use of that CPT test result for the computation of liquefaction potential. In CPT what we do? Instead of a thick-walled sampler, in this case we use a steel cone which is pushed instead of hammering it. So, we do not hammer it; we just push it in the soil. Different types of CPT can be there, mechanical, electrical, piezo type, etc.

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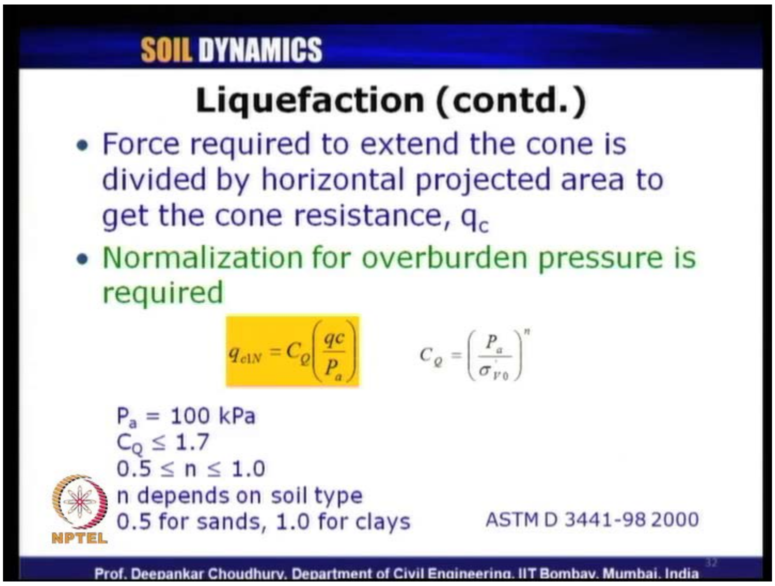

So, it is mostly suitable for clayey type of soil you know or fine grain type of soil. What are the things we measure from the CPT? The force required to extend the cone is divided by the horizontal projected area of the cone to get the tip resistance of the cone which is denoted by q c; that is how much load you require to push that cone in the soil divided by the projected cross-sectional area of the cone will give you the tip resistance. So, that is recorded here as q c and that q c also needs a correction factor. Because it also has to be standardized with respect to a particular overburden. So, here also overburden of one atmospheric or 100 kPa is considered as the standard overburden pressure. So, overburden correction factor is C Q.

So, the corrected value of q c 1N is equals to correction factor c q times the recorded value q c by P a; P a is the standard value one atmospheric pressure or 100 kPa and C Q is calculated as P a divided by sigma v naught dash to the power n. So, sigma v naught dash is effective vertical stress at the depth where we are calculating the liquefaction potential and n is a factor which depends on different soil type. It is 0.5 for sandy type of soil; it is 1 for clayey type of soil as per the ASTM standard number, this one. And N value lies between 0.5 to 1 for C FINES soil and here also we have an upper limit of C Q value of 1.7, this correction factor, this one.

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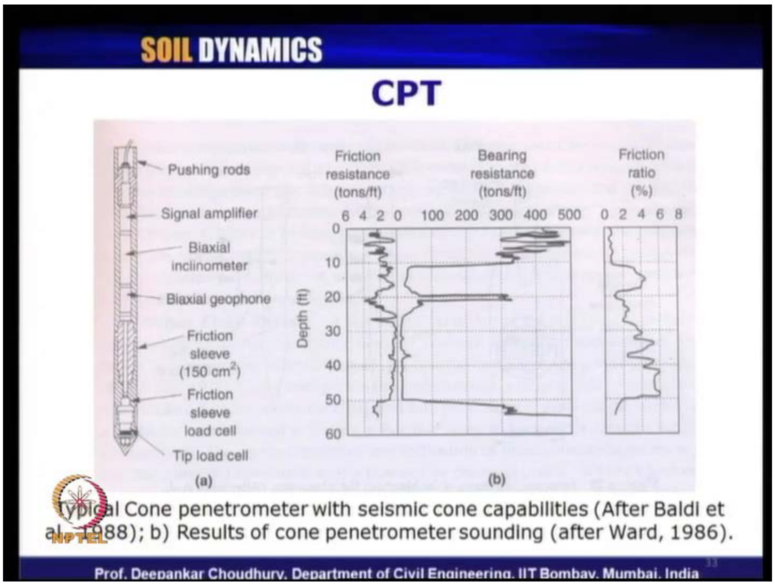

So, what is a general assembly of CPT cone penetration test? This is the picture typical cone penetrometer with seismic cone capabilities as shown here. Here this is the pushing rod, signal amplifier, biaxial inclinometer, biaxial geophone, here the friction sleeve, and friction sleeve load cell, and this the tip load cell. So, from this test when we are pushing this rod inside the mostly fine grained type of soil, what data we are collecting; with respect to depth we are getting the frictional resistance. This frictional resistance we can compute and at depth it is getting recorded like this. Also we are recording the bearing resistance or the tip resistance; the tip resistance is nothing but that q c value what I have mentioned just now. So, that also is given here. Then we can find out what is the friction ratio in percent.

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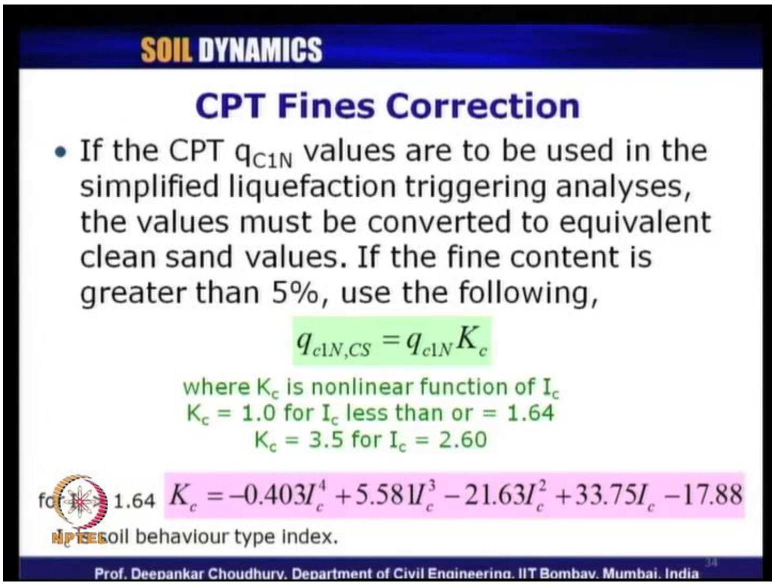

So, CPT fines correction also required for liquefaction analysis. Similar to our SPT fines correction; this is also required only for the liquefaction analysis not for other study or other design process, whereas the overburden correction factor is required for all other design process also. So, CPT q C1N whatever corrected value of cone penetration test value we got, after applying the overburden correction factor, that needs to be corrected for simplified liquefaction analysis in terms of equivalent clean sand.

Here, also the standardization has been made in terms of equivalent clean sand and if the fines content is greater than 5 percent, then we apply this correction factor. This is the corrected q c1N value in clean sand condition; K c is the correction factor. So, if percent fine is less than 5 percent again no correction is required. This K c is a non-linear function of another factor I c and K c is 1 for I c less than or equals to 1.64 and K c is 3.5 for I c equals to 2.60 and for I c greater than 1.64 that is in-between 1.64 and 2.6, the expression to compute K c is given by this equation where I c is nothing but it is indicative of soil behaviour type index. So, from Youd et al. again you can get the details of computation of each of these terms I c, K c, etc.

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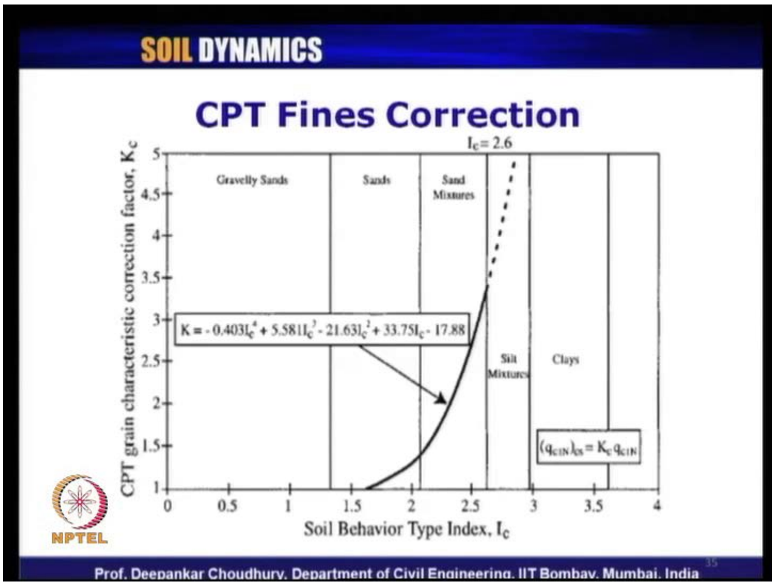

So, this is the chart for the correction CPT fines correction. What we have mentioned just now in terms of equation, the same thing plotted here in terms of design chart; that is y-axis is giving you the correction factor K c and x-axis is giving you the soil behavior type index I c. So, depending on type of soil, this zone is gravelly sand, this zone is for sandy type of soil, this is sand mixture, this is silt mixture, this is clayey type of soil, this is the curve. And equation what I have shown just now, that is for this curve linear portion and this is the correction factor as already have been mentioned.

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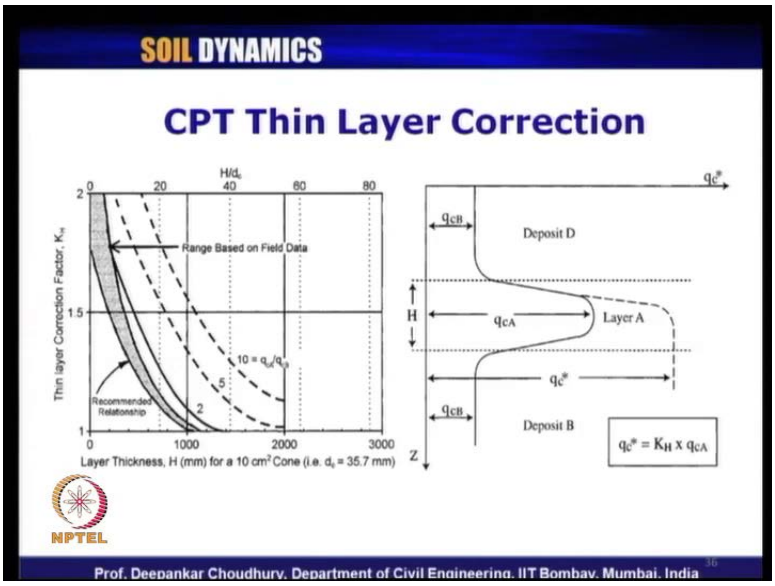

Now for CPT another correction is required which is called thin layer correction. What is thin layer correction? Look at this right hand picture first. Suppose we have three layers like this deposit D, layer A, and deposit B. In this case suppose layer A is a thin layer compared to this other two deposits B and D. So, in that case what is going to happen? Our recorded value of q c the tip resistance it keeps on recording as per the depth. So, this deposit and this deposit, suppose we got average value of this much; here also we got average value of this much. But in-between there is a thin layer which is very steep. So, suddenly there is an increase in the q c value. So, how to take care of the advantage of that q c increased value for a thin layer or whether to discard it; depending on that this thin layer correction is necessary.

So, that thin layer correction what it says; that q c star which is nothing but we can take this higher value by applying this correction factor K H in terms of q c A from this recorded value. And this thin layer correction factor K H is computed for H by d c value where layer thickness H is this one in millimeter for a 10 centimeter square cone which is having a d c of about 35.7 millimeter. So for different H by d c ratio, you will get what are the different correction factors, ranges based on the field data. So, this hedged portion is the recommended relationship what is proposed here to compute and use this q c value for a thin layer, presence of a thin layer a stiffer thin layer.

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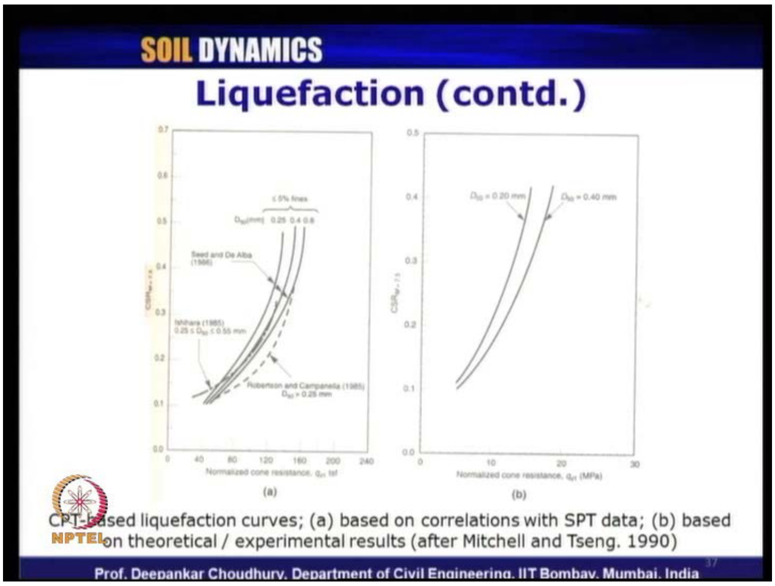

Similarly, what was done for CSR where m equals to 7.5 or the CRR the newly modified turn versus SPT value, the same thing also has been proposed by other researchers like Mitchell and Tseng, 1990, that CRR versus normalized cone resistance; normalized means after doing the all corrections, overburden correction and clean sand corrections, then this is the relationship for different fines content curve they also obtained.

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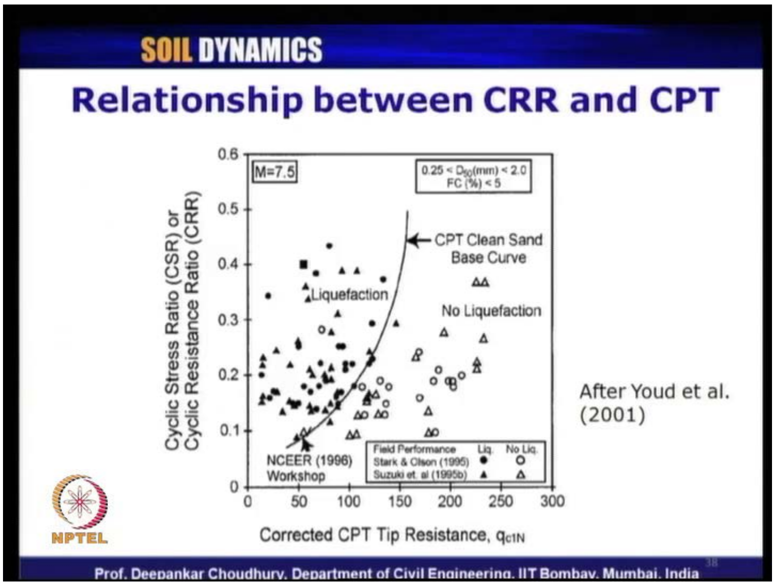

And this is the modified latest design chart which is commonly used given in Youd et al. 2001. X-axis gives you corrected CPT tip resistance q c1N which you have already calculated from your field measurement; go to that particular value, project your line here, go to this curve CPT clean sand curve, and then drop it on this axis this vertical axis which will give you the value of CRR for M equals to 7.5; that is the proposed curve.

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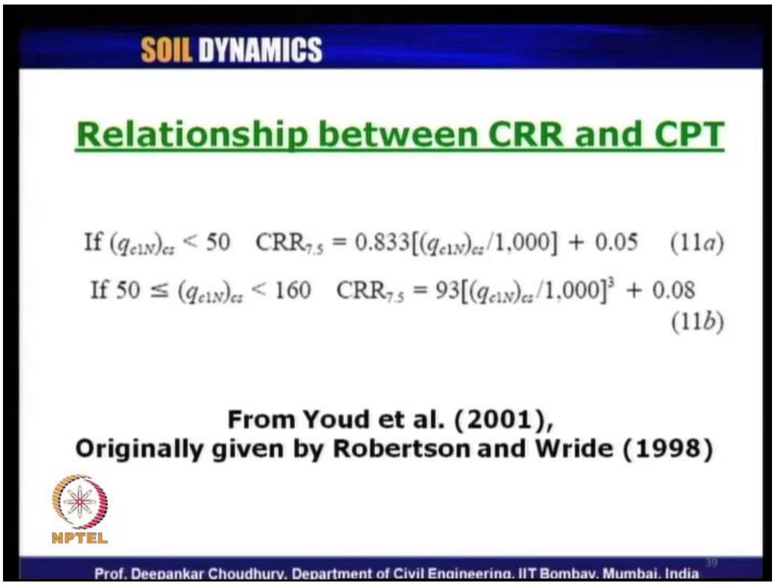

Other than that, like SPT you can use direct empirical relationship; that is either you can use the design chart to get the CRR value or you can use the empirical relationship to compute the CRR value based on your corrected q c1N clean sand value. So, what are those equations? Look at here; if (q c1N) CS is less than 50, then CRR 7.5 is computed by using this expression; if it is between 50 and 160, then the equation proposed is this one. It was originally proposed by Robertson and Wride in 1998 which is re-reported in Youd et al. 2001. So as a designer as you know, always it is advisable to go for this empirical relationship rather than using the designed chart because the chances of human error by reading the graph reading the design chart will be minimum, if you use this given empirical relationship both for SPT as well as for CPT.

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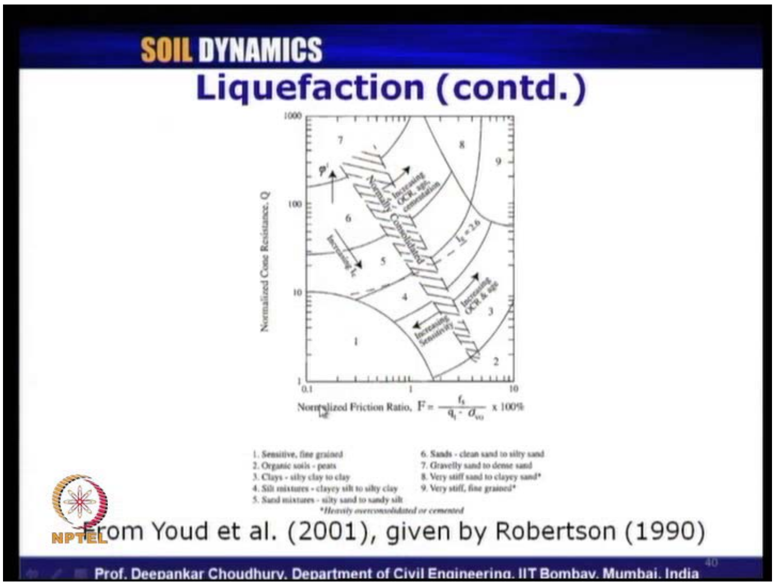

This is another chart which was given by Robertson in 1990, normalized friction ratio is calculated like this. The frictional force that sleeve friction whatever is measured divided by that q t or q c minus sigma v naught the vertical stress times 100 percent and what is normalized cone resistance can be obtained for different values of I c.

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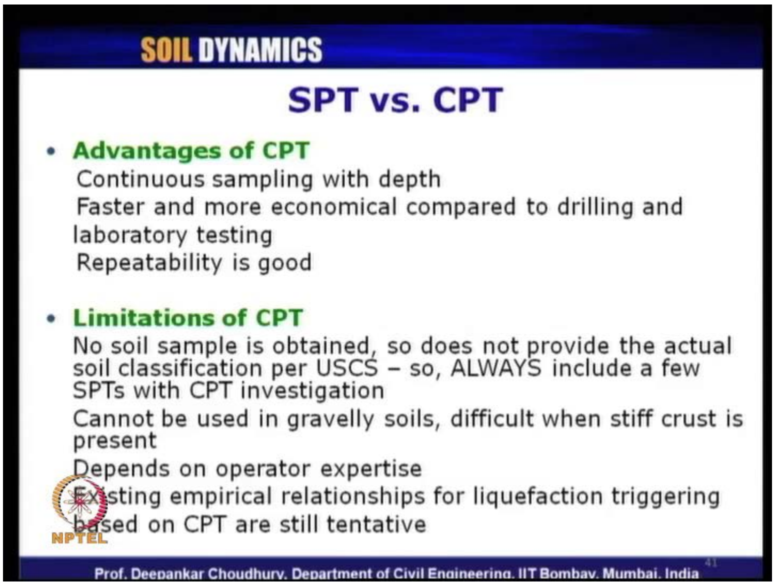

Now a kind of comparative study between SPT and CPT test results. What are the advantages of CPT cone penetration test over the standard penetration test? One is continuous sampling with depth; that is at each depth you are getting the result because q c and friction value also you are getting at each depth; you are penetrating the sampler tube through the depth. So, that is why you are recording keep on recording with respect to depth the value or the strength of the soil and it is faster and more economical compared to drilling and laboratory testing, why it is faster? Because just you need to push it in the soil and why it is economical; you need not to have a borehole and all these arrangements and hammer etc.

And your repeatability is good; repeatability means at the same side, suppose another person or another agency went and did the same CPT test they will also probably get the similar test results, whereas the repeatability for SPT test is not that good; it depends on your arrangements made by the different agencies, the testing procedure whether hammering is done vertically or incline eccentrically; all these factors comes into picture whereas those things are pretty less for CPT. So, that is why these are the advantages of CPT.

Whereas CPT is having some disadvantages or limitations also over the SPT test. What are those? No soil sample is obtained in the case of CPT because in CPT we are just pushing that rod inside the soil sample. So, nothing we are getting out of that below the ground; whereas in case of SPT, the SPT sampler tube which you are penetrating it, finally you are taking out the soil sample in the split spoon sampler tube. So, you get a fairly good estimate of actually what kind of soil is present. You can bring that soil sample and bring it to laboratory to do all the soil test. So, that is why SPT test is much more reliable till date because of its easiness or it is able to collect the soil sample which can be tested in the laboratory and you can have a direct feeling, you can see the soil whatever is existing much below the ground surface which cannot be seen so easily. So, that is why SPT is advantageous over CPT in that term.

So, it does not provide the actual soil classification as per USCS in case CPT. So, always include a few SPT with CPT investigation, like what we have mentioned for SASW test also you remember, there also we are not getting any soil sample. So, it is always advisable for this kind of test where we are not getting soil sample to have some few SPTs where you can able to see what is existing actually below the ground surface. It cannot be used in gravelly soil CPT because it is mostly for fine grained soil; otherwise you cannot push the rod, it is difficult when stiff crust is present obviously. The rod cannot go further down.

It depends on the operator expertise whether they are pushing it vertically or inclined way or not. Existing empirical relationships for liquefaction triggering based on CPT are still tentative, this is most important in our study; that is the relationship between CRR and SPT, the dots, the points, the field observations whatever has been collected and reported by several researchers, those are quite well established till date, whereas for the case of CPT it is not so well established like SPT in terms of liquefaction triggering analysis. So, it needs a further study.

End of Part A

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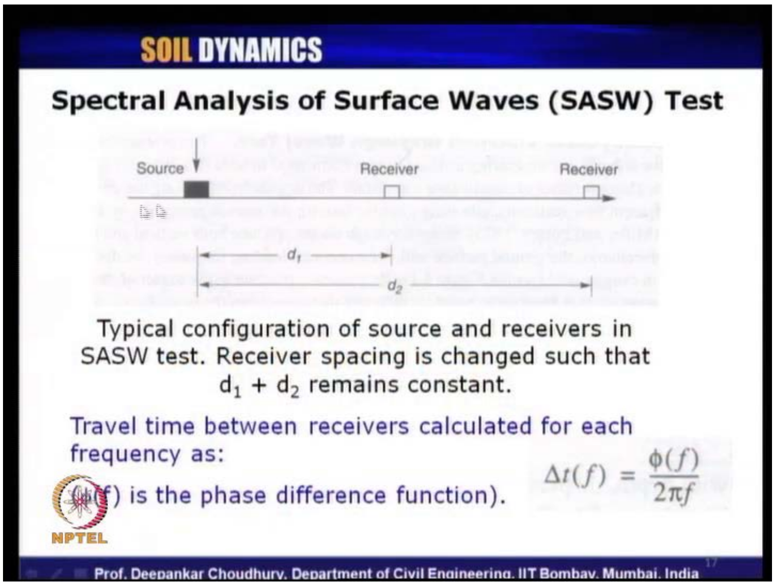

Now the another test which we had already discussed during our field test that spectral analysis of surface wave test or in short it is called SASW test; similar type of test as I have mentioned is MASW using multi-channel. So, this was the schematic diagram; we know about the basics of the test. Now how to use this SASW test data where we are recording the V s value shear wave velocity of the soil that is what we are recording.

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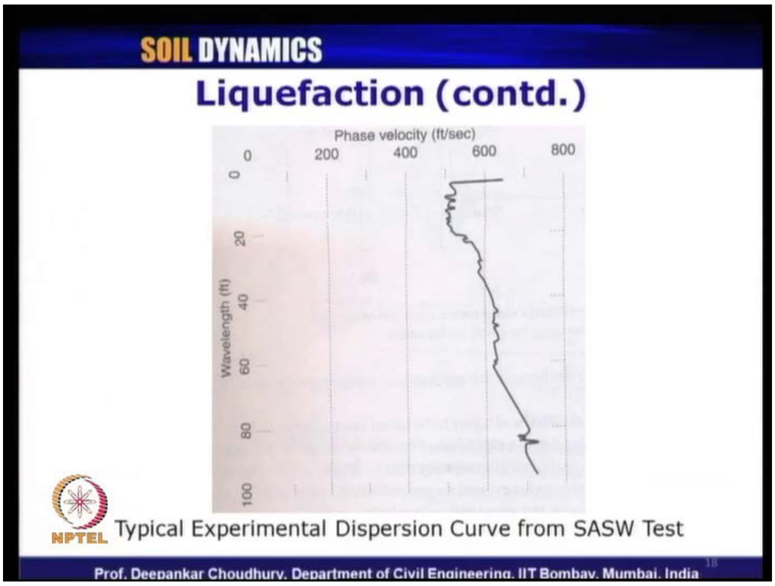

So, this is a typical variation of phase velocity with different wave length.

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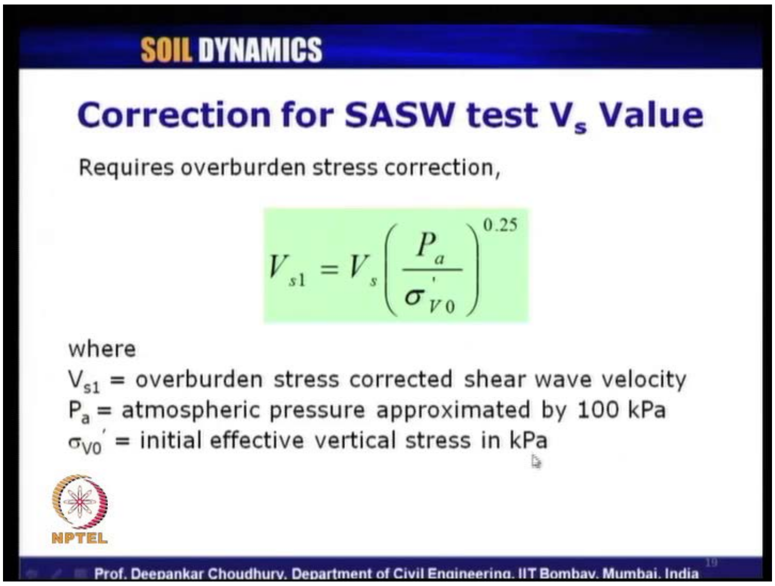

Now here also the correction is required; the overburden stress correction similar to SPT and CPT. So, this is the formula used to convert the raw or recorded V s value from SASW test to the corrected value V s1; the P a is again one atmospheric pressure at about 100 kPa and sigma v naught dash is the initial effective vertical stress in kPa unit. So, this is the formula we used to find out the corrected SASW value or corrected shear wave value after doing the overburden stress correction.

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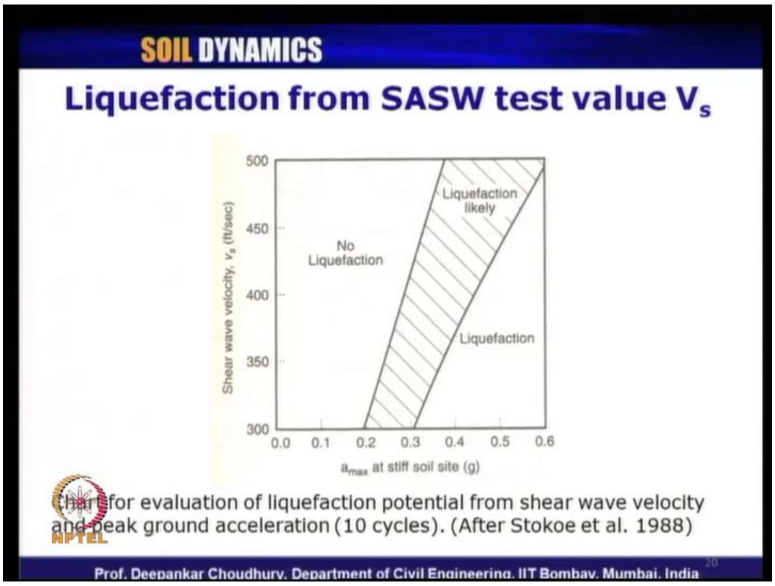

And liquefaction from SASW test value that V s, how they vary with different a max value at stiff soil site that is different peak accelerations during the earthquake 0.1 g, 0.2 g; these are the different ranges of acceleration. This chart was proposed by Stokoe et al. in 1988. It shows chart for evaluation of liquefaction potential from shear wave velocity and peak ground acceleration with more than 10 cycles in numbers. So, shear wave velocity is in feet per second unit, y-axis and x-axis is a max value in g unit; that is 0.1 g 0.2 g is the peak acceleration of the earthquake. It is found that generally in this region, soil will not liquefy; this hedge zone shows that it is most likely that soil may liquefy and this zone definitely soil will liquefy which is quite understandable that as the a max value increases; obviously, we will have more chances for liquefaction.

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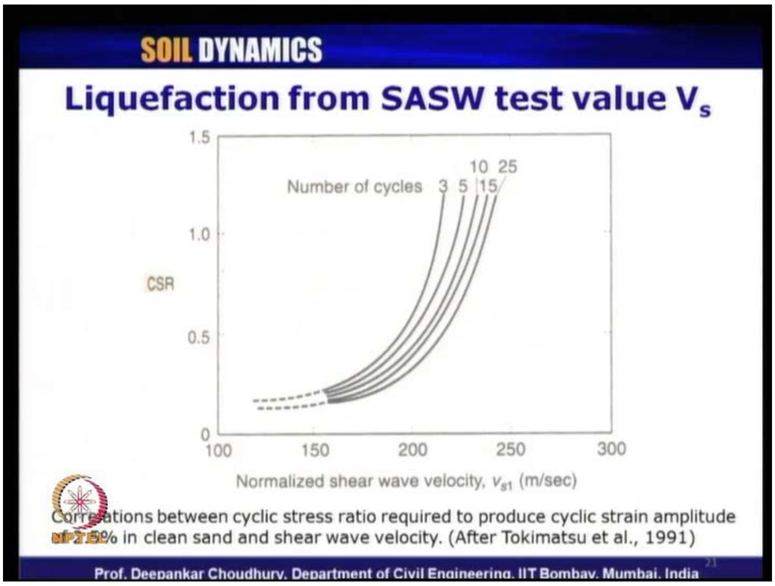

And the relation between the CSR versus normalized shear wave velocity V s1 in meter per second unit. Remember this CSR is CSR 7.5 as was earlier proposed by Seed and Idriss; that is in new terminology this is nothing but CRR. So remember that one, because this was developed before 2001. It was developed by Tokimatsu et al. in 1991; correlations between cyclic stress ratio required to produce cyclic strain amplitude of about 2.5 percent in clean sand and the normalized shear wave velocity. Normalized shear wave velocity means shear wave velocity after doing the corrections the corrected value of shear wave; depending on number of cycles of loading this is the typical variation of that CRR versus V s1 as was proposed by Tokimatsu et al. in 1991.

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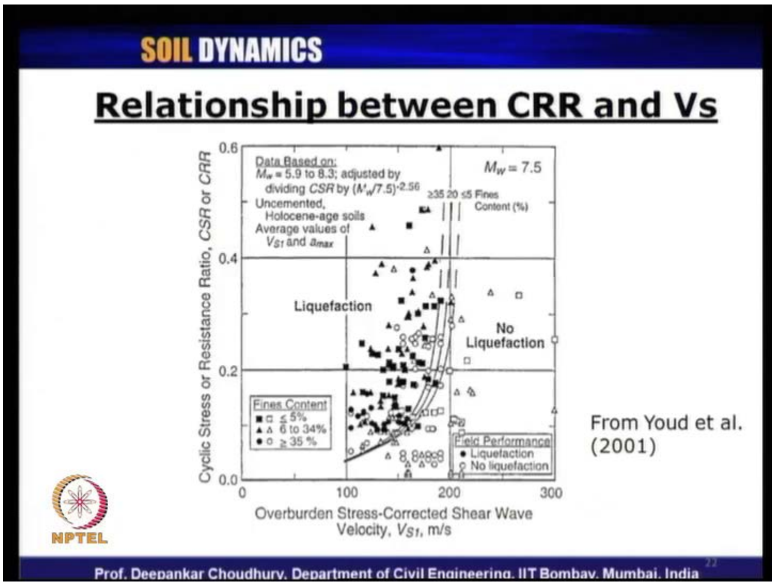

And what is the present status, what is used worldwide? This is the correlation between the CRR versus V s as given in Youd et al. 2001 for calculation of liquefaction potential using SASW value. So once you know your V s value, correct it for overburden correction. Then that corrected value you know this value here, project it based on your fines contents of the soil and then drop it on the y-axis you will get the value of CRR; that you will be again using for computation of factor of safety against liquefaction.

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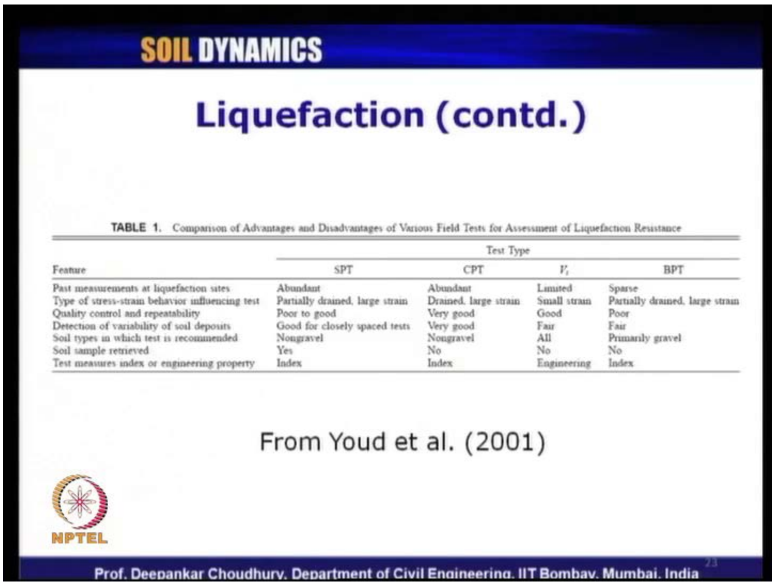
SOIL DYNAMICS

Liquefaction (contd.)

TABLE 1. Comparison of Advantages and Disadvantages of Various Field Tests for Assessment of Liquefaction Resistance

Feature	Test Type SPT	CPT	V_s	BPT
Past measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good for closely spaced tests	Very good	Fair	Fair
Soil types in which test is recommended	Nongravel	Nongravel	All	Primarily gravel
Soil sample retrieved	Yes	No	No	No
Test measures index or engineering property	Index	Index	Engineering	Index

From Youd et al. (2001)

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So, a comparative study as given in Youd et al. 2001 for different test types and different features regarding their advantages and disadvantages of this different field test for assessment of liquefaction potential. Suppose the feature we take past measurements at liquefaction site, then SPT is abundant, CPT also abundant, V s V s means it is using SASW test limited and BPT sparse; type of stress-strain behavior influencing test, then SPT partially drained or large strain, CPT drained large strain shear wave velocity which we are determining from SASW test; obviously, it is small strain analysis as we already know, and BPT also partially drained and large strain.

What about the quality control and repeatability? SPT gives poor to good results about the quality control and repeatability of results; means if you repeat that SPT test at the same sight, whether you are going to get similar n value or not that is what it means. In CPT it is very good; that we have already seen during the compression of SPT and CPT. SASW also good, whereas BPT gives about poor results. Detection of variability of soil deposits for SPT of course it is a good for this feature because you personally can see what soil exists in the ground. So, that is why good for closely spaced tests.

CPT very good, deduction of variability of soil deposits because at each depth, they are measuring; whereas for SASW test it is fair and for BPT is also it is fair. Soil types in which test is recommended, SPT is generally for non-gravelly type cohesion less soil basically for sandy type of soil or C FIVES utmost. CPT for cohesive soil it is good, V s for all type of soil it is good and BPT is primary for gravelly type of soil BPT is good. Soil sample whether it is retrieved means taken out during the test process or not; in SPT yes, we are getting the sample, we can see the soil.

In CPT no, we are not able to see the sample. In SASW test also no, we do not get the sample; in BPT also we cannot get the sample. Test measures index or engineering property; in SPT we are getting index properties. In CPT also we are getting index properties, whereas in V s we can get the engineering properties, and in BPT we can get index properties. So, with this we have come to the end of our module 4 that is dynamic of soil properties. So with this, we have come to the end of today’s lecture; we will continue further in our next lecture.