Geotechnical Earthquake Engineering
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Module - 9
Lecture - 41
Seismic Analysis and Design of Various Geotechnical Structures (Contd…)

Let us start our today's lecture, for NPTEL video course on Geotechnical Earthquake Engineering. For this video course, currently we are going through our module number nine, which is seismic analysis and design of various geotechnical structures. Before starting our today's lecture, let us do a quick recap, what we have learnt in our previous lecture. So, in the previous lecture, we started with the sub topic; seismic design of pile foundation, which is one of the most important area, in the civil engineering design and practice is concerned, for the pile foundation design under the seismic loading condition.

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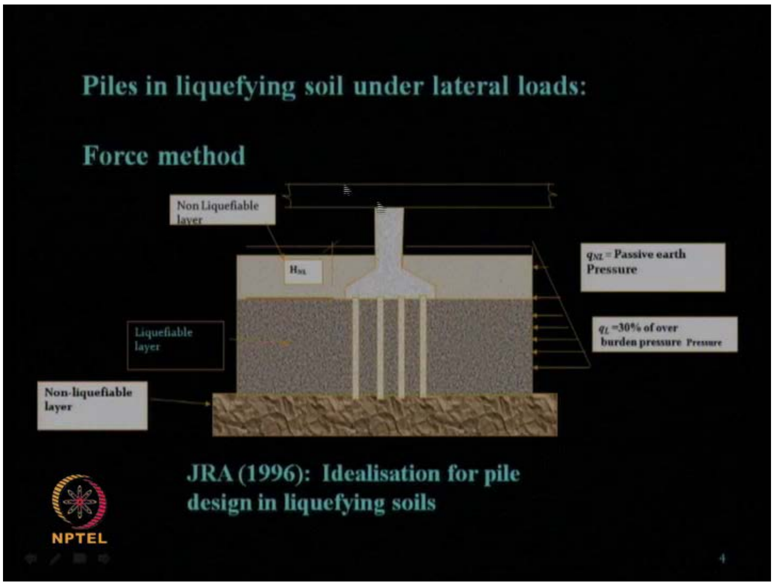

So, in that we have already discussed as you can see over here; that is piles, that is a super structure founded on this pile foundation, group of piles. They are subjected to extra lateral load, due to the seismic effect. So, seismic inertia forces etcetera, also if it passes through some liquefiable soil, during the earthquake, that will create additional problem on the pile foundation, which we have mentioned that through several cases stress. The examples of failures of piles are available, under the seismic condition, in liquefiable soil, as well as due to extra horizontal lateral load, which probably was not considered in the design. So, this is the basic recommendations as you can see on this right side by JRA 1996, idealization for pile design in liquefying soil, that is when the piles are going through some layers; initially non liquefiable layer, then liquefiable layer, and finally again non liquefiable layer.

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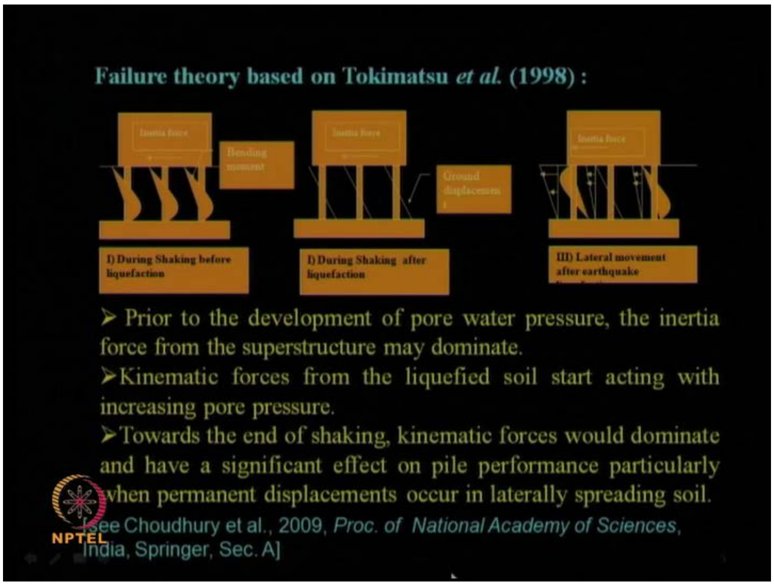

Now, Tokimatsu et al in 1998, proposed the failure theory of this pile foundations, under earthquake loading condition and liquefaction, how it occurs? So, this is the first step; that is the behavior of the pile, and this is the super structure during the shaking, and before the soil liquefaction occurs. So, piles are subjected to extra bending moment, because of this extra inertia forces, due to the horizontal load or inertia seismic load. And in the next stage; stage two what happens? the during shaking after the soil gets liquefied, the ground will get depressed, because soil gets liquefied, so ground displacement occurs. And in the third step; both of them gets combined; that means, this inertia force, and this is called the kinematic response of the soil, that gets combined and the combined effect on the pile will be its pile displacement and bending; that is lateral movement after earthquake due to liquefaction, due to this inertia forces.

So, the details about this responses or failure theory of Tokimatsu et al, is available in the review paper by Choudhury et al 2009, in the Proceedings Of National Academy Of Science, Springer publication Section A, physical sciences. Many other researches also had carried out the research, as I have already explained to you, in the previous lecture, in addition to that. Like Prof. Subhamoy Bhattacharya, he also did his PhD work, and also currently he is working with various of his students, who are working on this behavior of pile foundation under earthquake loading condition. Not only due to this lateral load, they consider the vertical load also in their analysis and consideration. Also Prof. B K Maheshwari from IIT Roorkee, they are also doing analysis of this earthquake conditions pile foundation, using fem approach that also are available in the literature. So, coming to case specific design for pile foundation, under earthquake condition, why case specific I has mentioned in the previous lecture, because as recommended by Poulos and various other researchers that, whether a particular soil location how much ground amplification, site response, soil behavior are going to effect the pile design, and seismicity needs to be considered, that is why case specific design is important.

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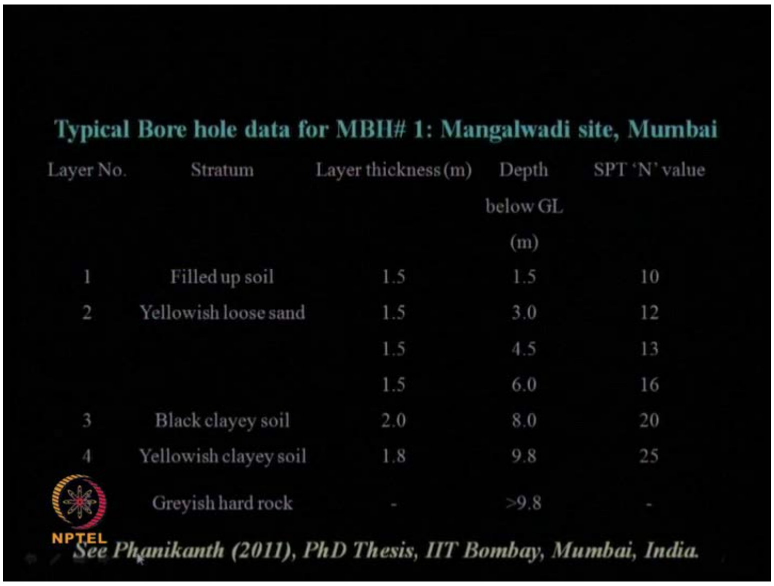

Typical Bore hole data for MBH# 1: Mangalwadi site, Mumbai

Layer No.	Stratum	Layer thickness (m)	Depth below GL (m)	SPT 'N' value
1	Filled up soil	1.5	1.5	10
2	Yellowish loose sand	1.5	3.0	12
		1.5	4.5	13
		1.5	6.0	16
3	Black clayey soil	2.0	8.0	20
4	Yellowish clayey soil	1.8	9.8	25
	Greyish hard rock	-	>9.8	-

See Phanikanth (2011), PhD Thesis, IIT Bombay, Mumbai, India.

For that I referred in our previous lecture itself, that the PhD thesis by Dr. V S Phanikanth, this PhD thesis 2011, at IIT Bombay. He did PhD under my supervision at IIT Bombay. So, initially he carried out the ground response analysis, from various soil sites at Mumbai city. This is Mangalwadi soil site, bore hole number one, typical soil input data.

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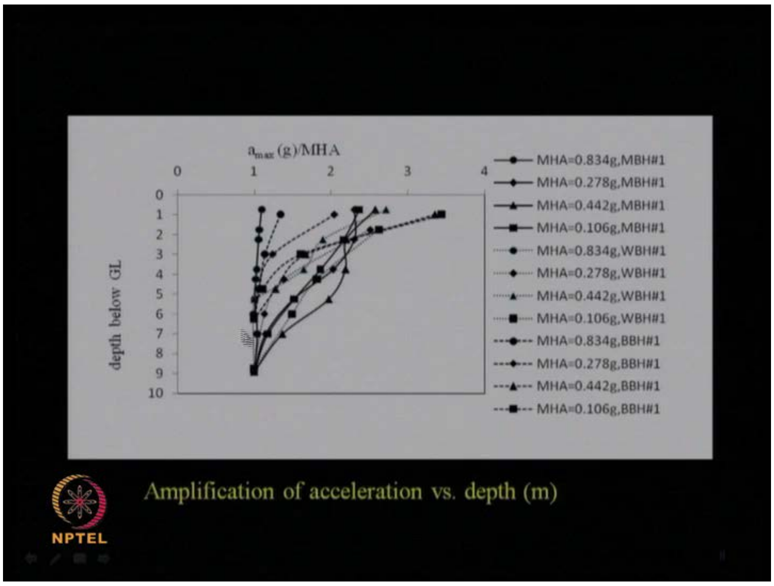

We are getting from the borehole data; that has been used under different earthquake loading conditions, using different input motions like; Cobe Input Motion, Loma Prieta, Loma Gelerian, Bhuj Earthquake Motion to find out, how much amplification from the bed rock level to ground surface, through various layers of the soil, how much amplification of the soil is going to get, how much amplification of the seismic response or acceleration is going to occur, due to the soil properties, through the ground response analysis using deep soil.

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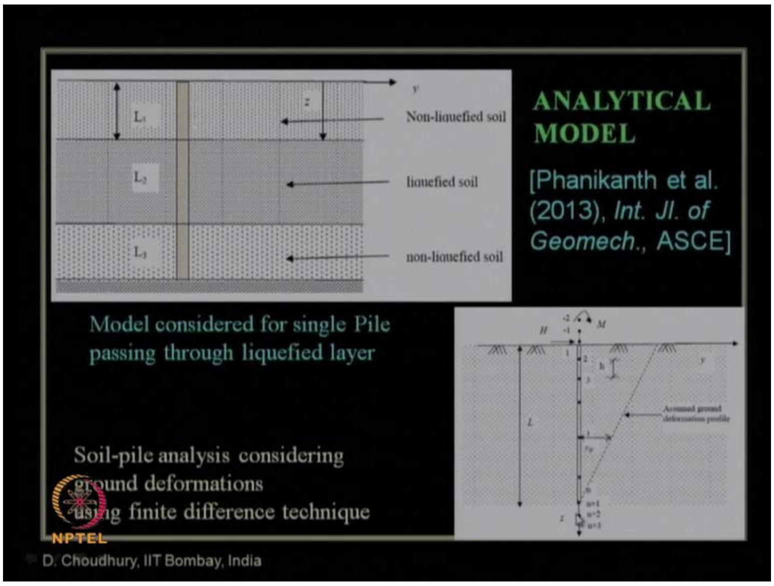

Now when we were discussing the pile analysis, Dr. Phanikanth had chosen a single pile analysis, behavior of single pile which is passing through suppose these three layers; one is non liquefiable layer, this is liquefiable layer of thickness L 2, and the again non liquefiable layer of thickness L 3. So, this parametric variation of the ratio of this L 2 over the entire length of the pile L, has been taken care in this analysis, using the concept of finite difference method. So, in this finite difference method, the entire pile section has been subdivided into n number of section, from the assumed ground profile using Winklers Beam Theory.

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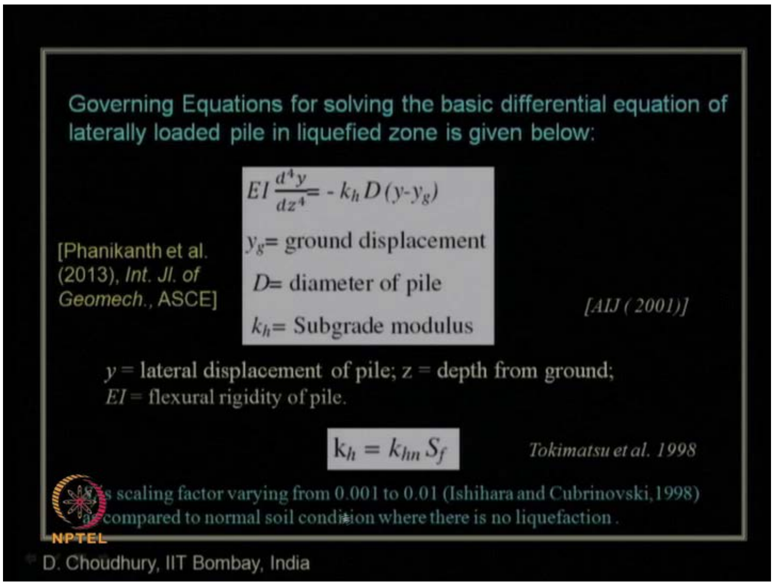

The basic concept of basic governing equation was used for pile in liquefiable zone, proposed by this equation. The details are available in this journal paper Phanikanth et al 2013 in the International Journal of Geo-Mechanics of ASCE USA. So, how this sub grade modulus is changing under the liquefiable condition of the soil; that also has been proposed by Tokimatsu et al in 1998, and values of this scaling factor are proposed by Ishihara and Cubrinovski in 1998, which have been used for the analysis.

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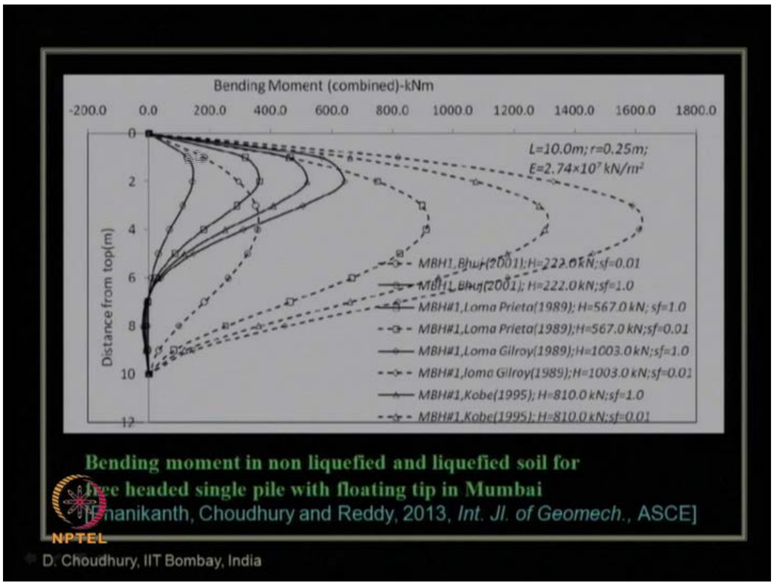

And finally, the bending movement of the pile, this black solid line shows the bending movement and the non liquefiable soil, that is when soil is not liquefying. And these dotted lines are showing the bending movement under the liquefiable condition. So, we can see there is a huge increase or significant increase in the bending movement of the pile, along the depth of the pile, from non liquefiable soil to liquefiable soil. And these are the reasons why many cases, in practice we see failure of piles after an earthquake is accruing, and the soil gets liquefied. So, the details will be available in this paper.

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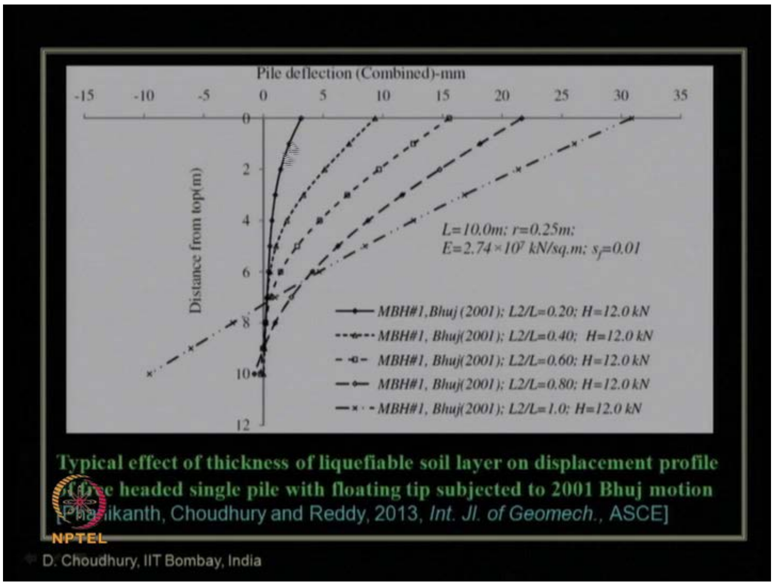

Also the pile deflection, we can see as a ratio of this liquefiable layer to the total length of the pile. It is changing or increasing from 20 percent to 100 percent, you can see the pile deflection is also increasing like this. Now, in today's lecture, let us start with another subtopic on this area, which is combined pile raft foundation, which is in short it is called CPRF, combined pile raft foundation, under earthquake condition. So, before we address this combined pile raft foundation. This is a very advanced topic and very recent one. It is still getting developed around the world, and people are doing research in this year of 2013. So, this is a very present hot research topic across the world.

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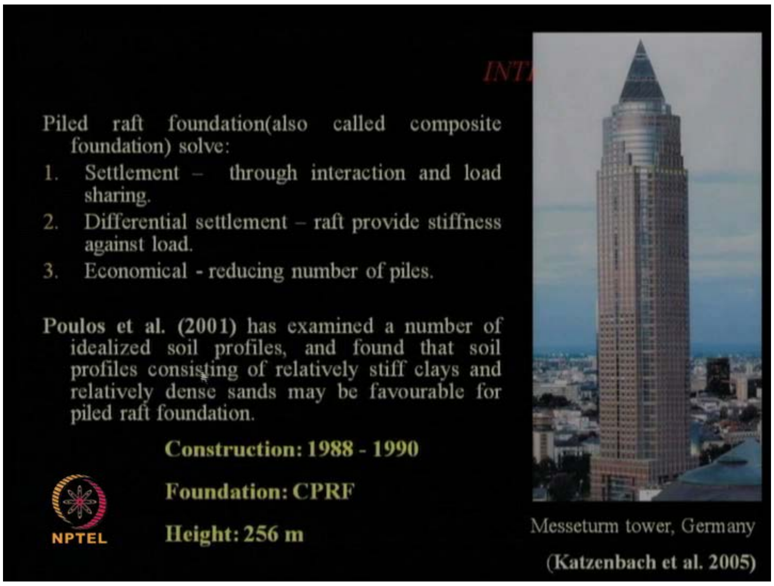

So, first to understand what is the pile raft foundation, let us go through this. This pile raft foundation is nothing but also called it is a composite foundation, because the pile along with the raft; that is most of the time as we know, pile above there will be pile cap. If you design that pile cap properly, to take a shear of the loading, and the response behavior coming from the soil, because of the soil pressure below the raft. So, that combined action of the raft and pile is nothing but the pile raft foundation CPRF. So, its settlement occurs through interaction and load shearing, between the two sections differential settlement like raft provide the stiffness again load, and it is very economical, why it is very economical. Because in this case you are considering the effect of raft also, which is neglected in case of only pile foundation; that is we always put the pile cap, but we never take the effect of pile cap. So, that is why, using this pile raft combined design approach. You can get the reduced number of pile, which are required for a particular loading.

So, that is why it is very economical. And the one of the pioneering work in this pile raft foundation as you can see in this picture. This is nothing but the Messeturm Tower in Frankfurt am Main in Germany. So, this foundation, this is founded on pile raft foundation, again this work has been carried out Prof. Katzenbach et al, Katzenbach in his research group. They are doing this work since two decades, and you can see over here this construction of this Messeturm Tower was done in the year 1988 to 1990, foundation was CPRF. Height of this building is 200 and 56 meter. Then Poulos et al in 2001, he and his research group also has explained a number of ideal soil profile, and founded that soil profile consisting of relatively steep clays, and relatively dense sand may be favorable for pile raft foundation. You should remember there are conditions, which are required for this CPRF. It is not that in all cases you can use CPRF. There is a particular ratio of the stiffness of the soil layers from the, raft level to the pile bottom level, it should be within some range then only you can apply this combined pile raft condition.

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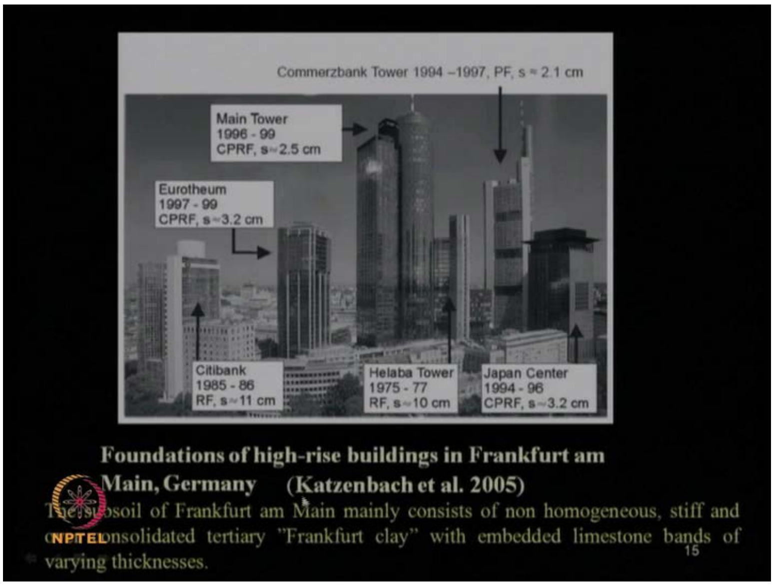

Coming to next slide you can see over here, in Germany in Frankfurt am Main, there are several high risers which are founded on the pile raft foundation, and the entire work was done by Prof. Rolf Katzenbach, who is the Prof. at Technical University Darmstadt in

Germany. And he and his research group are working as I said, since last two decades last more than two decades, in this area of combined pile raft foundation. And he is one of the pioneering and well established researchers, who work extensively and worldwide in this area of CPRF.

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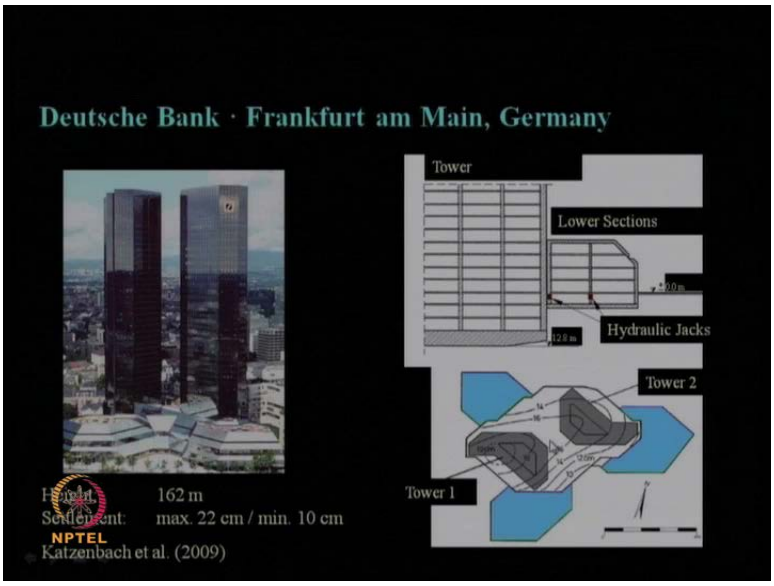

Another example like deutsche bank head office, which is again in Frankfurt am Main in Germany; that is also, there are two towers upper tower and lower tower portions; that is also was going through excessive settlement, and that was stopped using this hydraulic jack technique etcetera.

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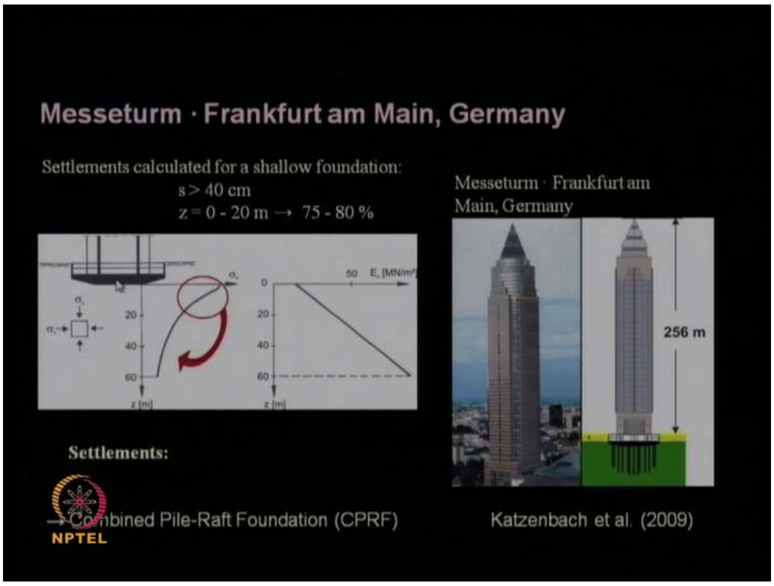

Then why for Messeturm Tower this CPRF technology was recommended. If you see, suppose if somebody uses the, only raft type of foundation for this type of soil. Remember in Frankfurt am Main typically it is a soft clay, it is called Frankfurt clay. Up to a depth of it is varying between 15 20 meter, up to even 100 meter also. So, this is the range of the stresses going to get established, if somebody uses the raft foundation.

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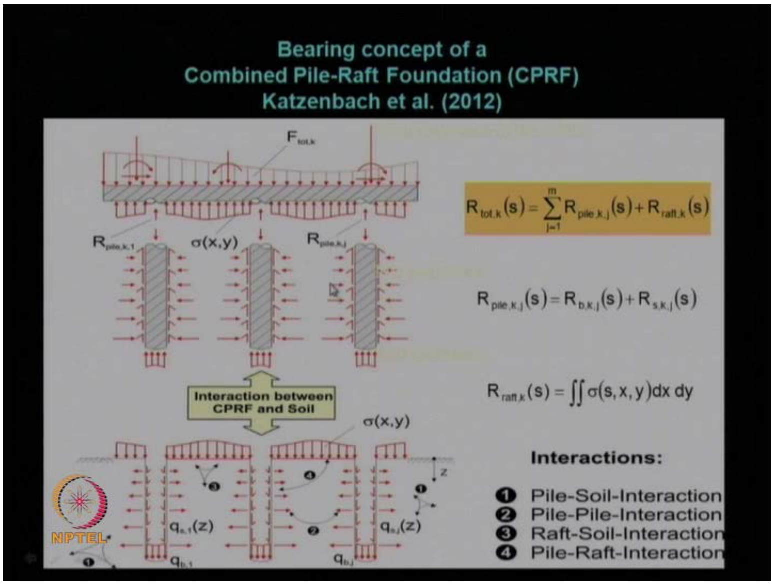

And settlement will be huge, that boils down to use of the combined pile raft foundation. So, combined pile raft foundation as I was mentioning, raft is not possible, because it gives us huge amount of settlement. So, to reduce the settlement you should go for deep foundation, which is pile foundation. And what is the pile foundation, the problem is it will be very costly, because number of piles, when you are using more number of piles, it will be costly. So, somewhat in between solution is always this combined pile raft, if your soil profile and site condition is preferable. So, those are very important guide line and criteria, which is discussed in this publication Katzenbach et al 2009. I am also a co author in this paper; we did an extensive research in this area as well, during my visit as a humbled fellow at Technical University Darmstadt, to work with Prof. Katzenbach and his research team.

So, to reduce the settlement; this excessive settlement, and to reduce the cost, the intermediate solution or best solution I should say, is the combined pile raft foundation for a chosen soil condition. So, this is the basic concept of bearing, what it occurs in the CPRF. So, the detail of this analysis will be available in the Katzenbach et al 2012. This is another paper, where I am also involved as a co author. So, this paper in this conference paper is available. You can see over here the infraction. This is the raft portion. This is the raft and soil interaction pressure. These are external loading, loading and moment etcetera. These are pile; you have the pile response from skin friction as well as end bearing. Now, infraction between the CPRF and soil, how it occurs? You have this portion soil response, and this portion soil response. So, you are now considering this raft portion pressure, as well as pile portion pressure. So, in pile foundation design only the pile foundation this part is neglected.

The part of the raft is not taken care of. So, what will be the total capacity of the loading; that is nothing but some of all these responses coming from individual pile. So, if you are using m number of pile you are getting m number of responses, and this is from the raft. Also for each file your having two components; one is end bearing, one is skin friction. And for raft, how you are getting it, you are getting it by integrating it, over the area for this stress in the x y plane. So, what are the interactions occurring, this zone 1, zone 2, and zone 3, zone 4. Zone 1 represents the pile soil interaction, zone 2; this represents pile to pile interaction, one pile to another adjacent pile. Zone 3; represent this zone, raft to soil interaction, and zone 4; this one represents pile to raft interaction. So, this is a nothing but a soil structure interaction problem, or I should say, soil pile soil or soil structure pile interaction problem, in a complicated manner involving both raft as well as pile, through this process. So, in this case simple Winkler Beam approach will not work properly.

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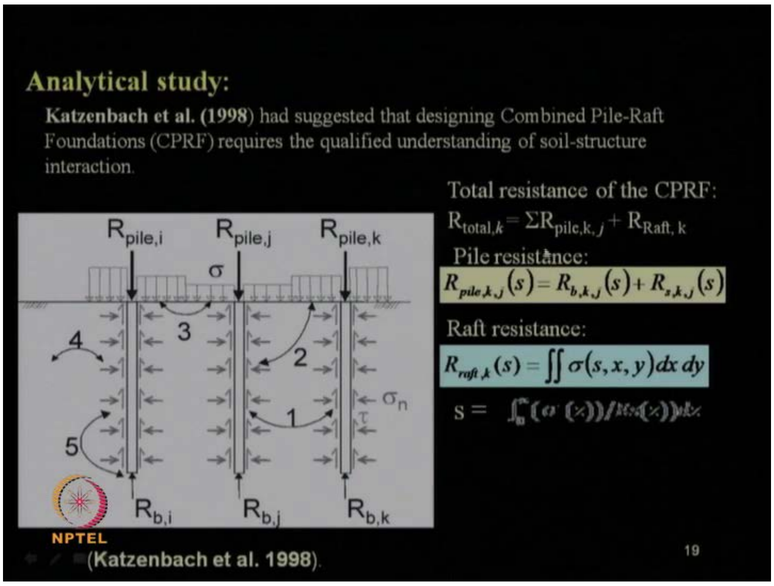

So, this is the analytical study, as proposed in way back by Katzenbach et al in 1998. They suggested that designing this combined pile raft foundation CPRF, requires qualified understanding of soil structure interaction, using this steps.

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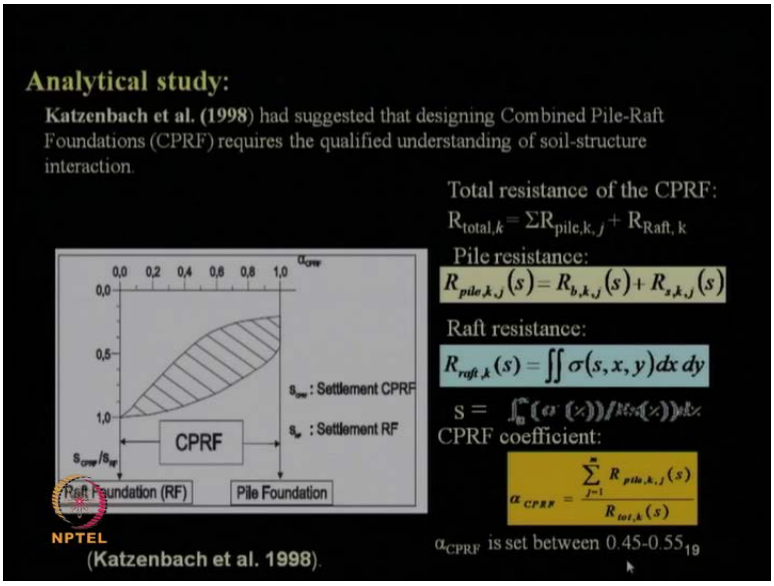

And now, we are going to introduce one parameter which is called CPRF coefficient. What is CPRF coefficient, this is nothing but the load shared by the total number of pile, divided by total load; that means, how much fraction of the pile is taking the entire load. It will be best designed, if it is 50 percent or 0.5. This CPRF coefficient is 0.5 means, you have designed it best; that means, your raft is sharing 50 percent load, and pile is sharing 50 percent load. So, we suggest for better design, the range of CPRF is given 0.45 to 0.55; that is, the typical range of this CPRF, is set always for a better design of this CPRF foundation.

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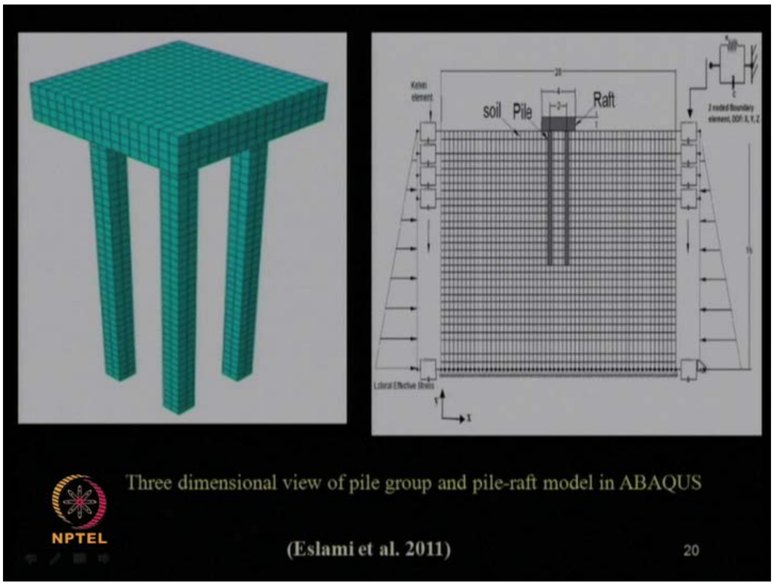

Other researches also, recently have taken up this important topic of research as I said. You can see, the combine pile raft foundation has been modeled in abacus software. The three dimensional analysis was done by Eslami et al in 2011, under earthquake condition. So, this is the node condition you can see, the damper, stiffener and the mass using this links.

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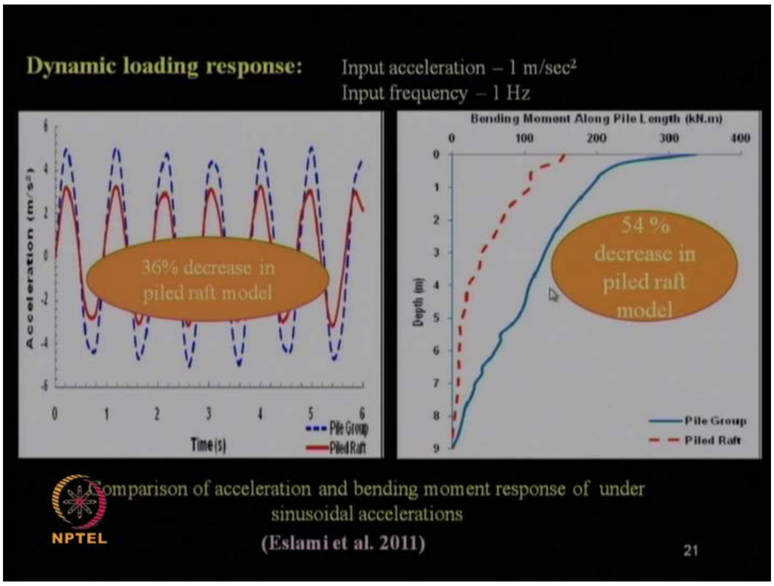

And the dynamic loading response you can see over here, as proposed by from their dynamic analysis with input acceleration of one meter per second square, with input frequency there is a 36 percent decrease, in the pile raft model compared to the pile group. That means, automatically that pile raft performs much better than a pile group under the dynamic loading condition as shown by these researches. Also you can see here, pile group bending moment and pile raft bending moment. Why it occurs, because here the load shearing between the raft and file is going to take place, also in the dynamic case as well. These researchers mentioned that 54 percent decrease, in the pile raft model of the bending moment, using this CPRF technology under dynamic loading condition

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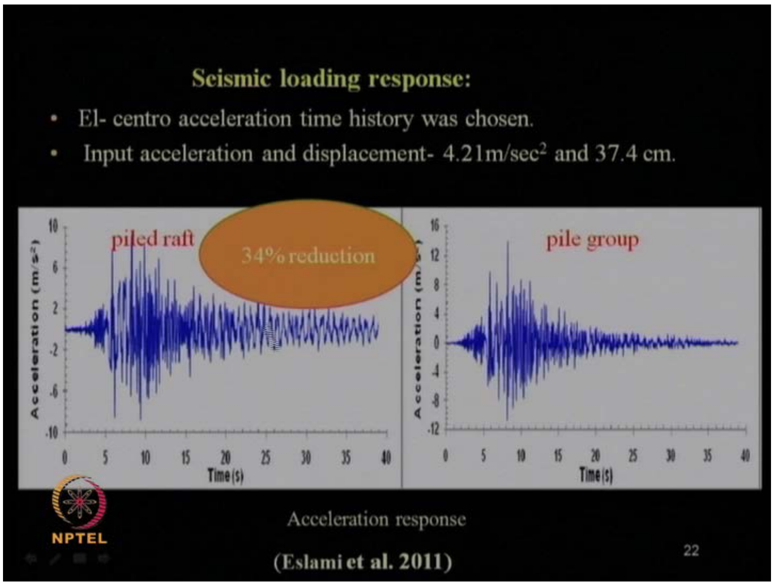

Also Eslami et al 2011, they used the seismic loading condition also. Earlier was dynamic loading, for seismic loading the input acceleration and displacement was considered as given as this values. And El-centro earthquake, very well known El-centro earthquake acceleration time history, they have chosen as the input acceleration time history result. You can see over here, they mentioned from their results that there is a thirty four percent reduction in the pile raft, in terms of acceleration time profile or response compared to your pile group response.

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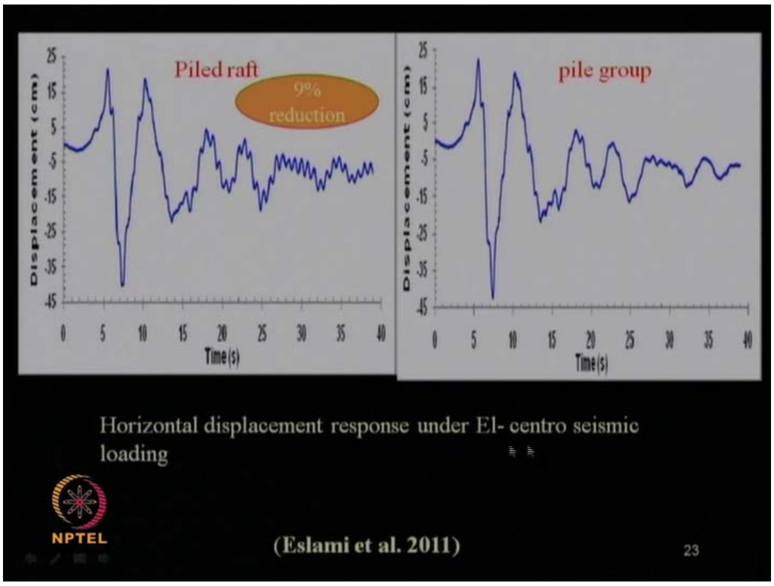

And this is the displacement amount, there is a 9 percent reduction in the pile raft compared to pile group, even with El-centro seismic earthquake loading. Now, we will discuss one case study, for this CPRF technology, which has been observed very recent 2011 Tohoku earthquake, after Tohoku earthquake in Japan. So, very recently, the researchers coming out, with this case study results. This is very important, because from this case study we learn, how this CPRF is functioning under earthquake conditions.

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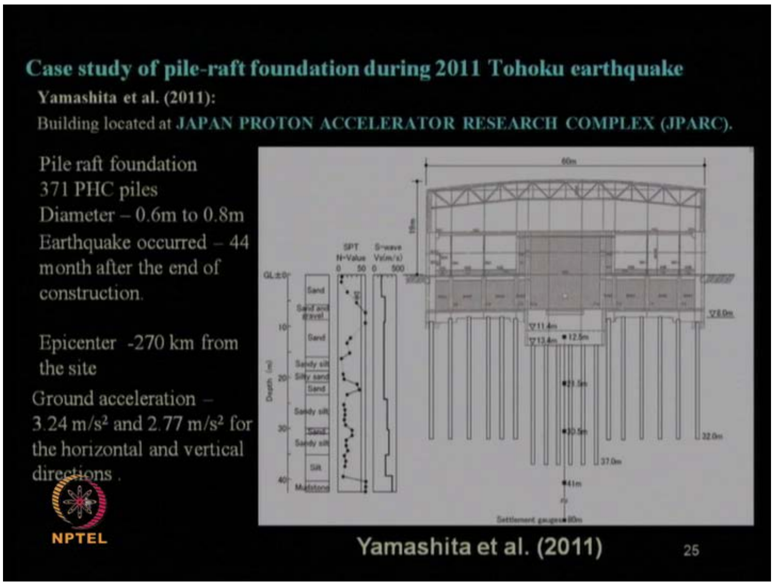

So, you can see over here this researchers Yamashita et al 2011, they proposed the case study of pile raft foundation behavior during this march 2011 Tohoku earthquake in Japan. This is the building located at the Japan, proton accelerator research complex JPARC. The data is like this, pile raft foundation, this pile raft foundation. This is the soil profile, you can see over here profile of the soil with SPT N value shear wave velocity profile along the depth, of that region. So, that ground response analysis, everything was done. So, it was pretty safe. So, let us see the pile raft foundation how it behaves. It was constructed on 371 PHC pile. The diameter is given over here. And earthquake occurred 44 months after the end of the construction of this JPARC building. And epicenter was about 270 kilometer from this site, where this structure was constructed with this CPRF. And ground acceleration value was this and this for horizontal and vertical direction.

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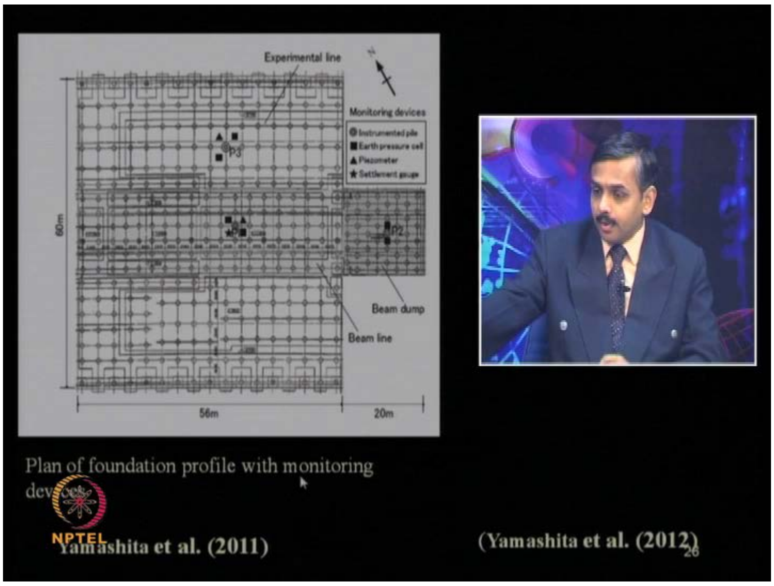

And what are the recorded response, that is they have used in that CPRF, because even worldwide nowadays people are using in CPRF, to instrumented piles, so that they can get the behavior of the pile; that is deflection and bending moment behavior from this instrumented pile, at collected from the field data or field site condition. So, they had instrumented pile like; pile p 1, p 2, p 3. These are instrumented pile, earth pressures instrumented pile, earth pressure cell and piezo meter, and settlement gauge, everything were provided at that site.

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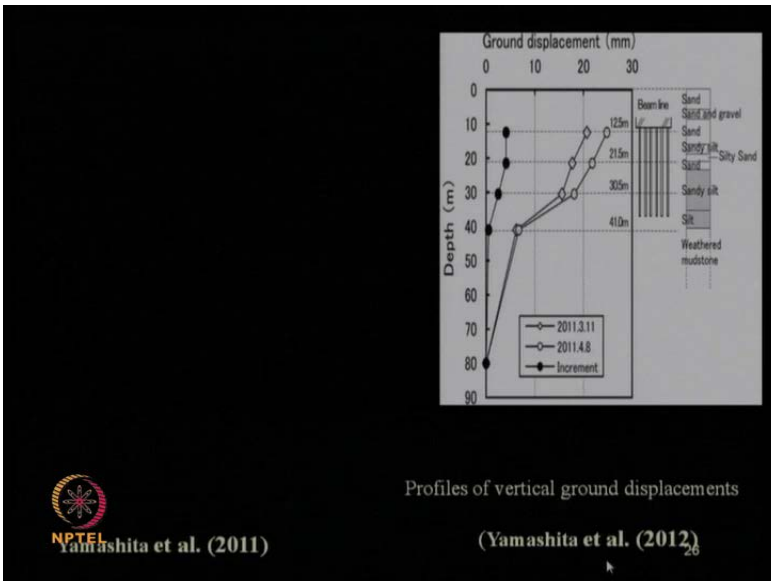

And this data shows the variation of results. This is the day of 11th march 2011; that is the day when the Tohoku earthquake occurred, can you see this data. And after the earthquake occurred, after about close to a month time; that is on 8th of April, 8- 4-2011, this is the Japanese way of writing date. They write year then month then date. So, this 8th of April 2011, this much displacement of the pile had occurred. So, increment is this much only, can you see. That means, CPRF what was designed to take care of the earthquake loading at the site. They performed very well at that huge damaging earthquake of Tohoku earthquake. The data are available in the paper by Yamshathi et al 2012, IS Canazava conference paper.

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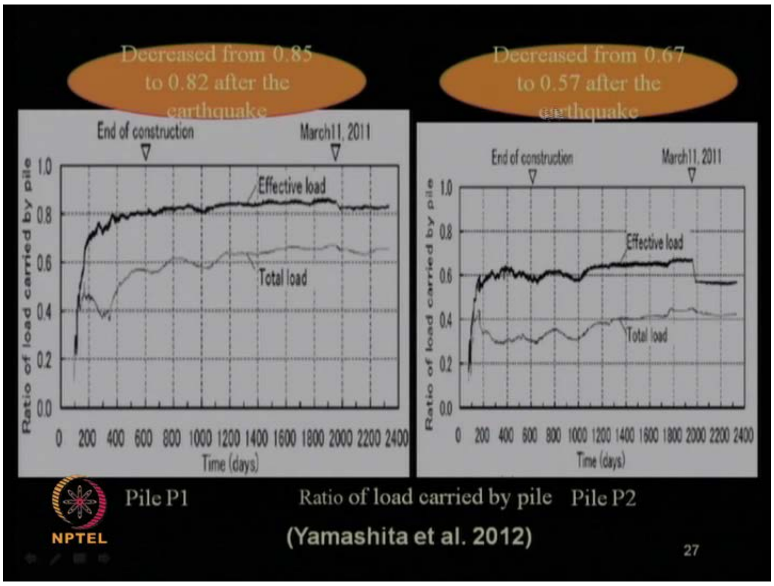

So, these are the responses, as I was mentioning for pile 1; p 1, this is the total load response, and this is the effective load response. This was the time in days in the x axis, and y axis shows, the ratio of the load carried by the pile. So, at the end of the construction, you can see it is almost saturated. This is the time when the earthquake occurred March 11, 2011. And this is the ratio of the load sheared by another pile that p 2, which is also an instrumented pile as I had already shown. So, from these things, it showed later on by Yamashathi et al in 2012 paper; that there is a decrease from 0.85 to 0.82 after the earthquake. There is a very marginal decrease you can see, by pile p 1 after the earthquake, the load shearing. And decrease from 0.67 to 0.57 after the earthquake by pile p 2.

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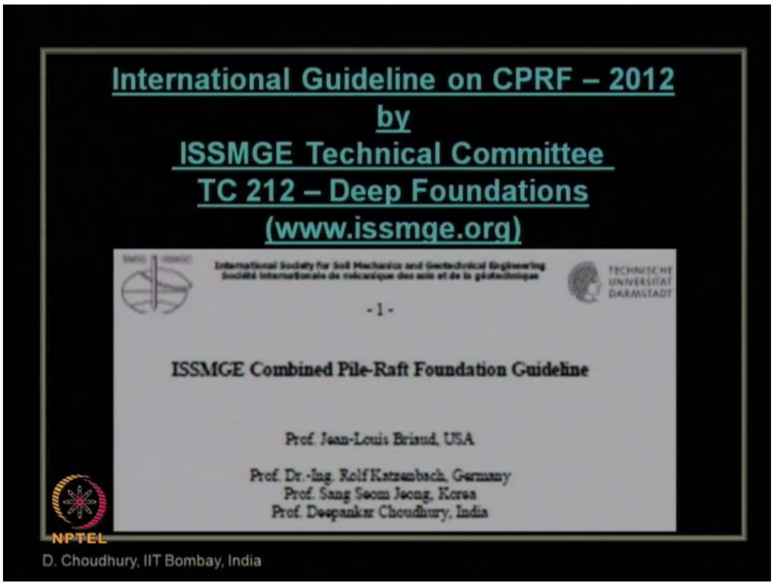

Now let me tell you, very recently international society for soil mechanics and geotechnical engineering, which we called ISSMGE. This ISSMGE is nothing but international society for soil mechanics and geotechnical engineering. We, that technical committee TC 212, which is on deep foundations, we came out with international guideline for this design of CPRF in very recently in 2012, during our committee meeting at IS Kanazava in Japan, in September 2012. This is the version of the ISSMGE combined pile raft foundation guideline, which has been proposed by this technical committee of ISSMGE on Deep Foundations. And present president of ISSMGE is Prof. Johnl'hebreu. And a present chairman of this TC 212, Deep Foundations Technical Committee is Prof. Rolf Kerzenbach from Germany. Vice chairman of this TC 212 is Prof. Sang Young, from Korea. And currently, I am the Secretary of this ISSMGE Technical Committee TC 212 Deep Foundations, myself Prof. Deepankar Choudhury form India.

So, all our members together after our meeting, we have finalized this international guideline; that is whoever wants to design any combined pile raft foundation; they can follow this design recommendations or codal recommendations or design guidelines. The international design guideline given by this technical committee worldwide, that these are the steps to be followed for design of a combined pile raft foundation in practice. (No audio from 26:38 To: 27:01). Now, let us go to next subtopic for this module, which is seismic design of ground anchors. And for this topic, I will like you to note down, this is the reference which I am using, let us look at it. This is the PhD thesis of Dr. Sunil Rangari, who completed PhD at IIT Bombay, under my supervision and Prof. Diwaikar supervision at IIT Bombay, in this year 2013. So, from his PhD thesis work, we will discuss about seismic design concept of ground anchors.

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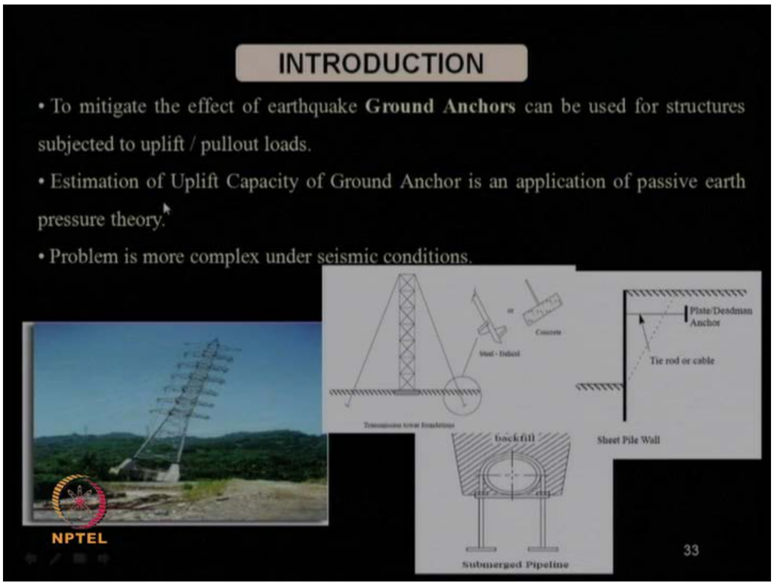

Now what are the ground anchors, as we all know where we use ground anchors, like to mitigate the effect of earthquake. Ground anchors can be used for structures, which are subjected to uplift or pull out type of load. So, which are those structures; like this transmission towers, chimneys, tall buildings. So, these are always subjected to some kind of pull out load or tensile load or uplift load. So, to protect the foundation against those pullout uplift, we provide the ground anchors, so that it remains in its place, when the anchors are provided. So, that is why this anchors need to be designed properly, so that it can withstand the pullout load, including the uplift coming from the earthquake forces.

So, estimation of the uplift capacity of this ground anchors. It is nothing but an application of the passive earth pressure theory. So, passive earth pressure theory as we have already explained in one of our previous lecture; that for positive wall friction angle, for the case of passive earth pressure behind rigid retaining wall; that is the example of bearing capacity factors for shallow foundations; that we have also seen under the earthquake condition, how to estimate the bearing capacity factors, under seismic conditions. Now the same passive earth pressure theory, but for negative wall friction angle case, is nothing but the application for ground anchors. So, that is why when we are calculating the uplift capacity of ground anchor, I can explain you through this basic diagram, as I have earlier also mentioned.

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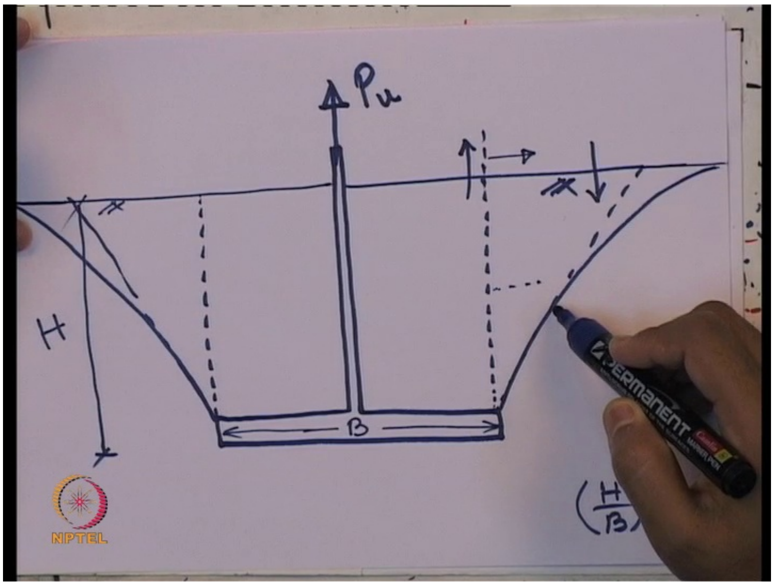

Suppose, this is a anchor rod, which is connected to an anchor plate something like this. Say this is this ground anchor is embedded up to a certain depth. Let us say this is the depth of embedment of the anchor H, and this is the width of the anchor plate. Now there are various types of anchors of course. So, this is plate anchor, and for plate anchor, we can have shape of square plate anchor, rectangular plate anchor, circular plate anchor like that. And even strip plate anchor also, strip means when length is much more than this width. So, this anchor rod along this, there will be some uplift load. So, this is the p uplift; that gives us the capacity of the anchor. Now, how to determine this capacity of a particular anchor, which is embedded in some soil like this. And there can be different types of anchors; one is shallow anchor, and another is deep anchor.

Shallow anchor, that depends on the value of this H by B, which is known as embedment depth. So, this H by B is an important parameter, which is called embedment depth, based on that value it is decided whether it is a shallow anchor or it is a deep anchor. Now what will be the typical failure surface for this type of anchor, ground anchor, when it is subjected to this uplift pullout like this. So, typically it can fill like this, this is one of the typical failure mechanism, or it can fail like this; that is when we are trying to pull it out, like this in this direction. Obviously, the soil about it will push out like this. So, we need to consider, let us say this we are considering as, an imaginary wall, imaginary retaining wall. Actually there is nothing it is within the soil only we are considering it as an imaginary wall.

Now what will be the movement of this wall laterally, if we consider this. This wall is actually moving, towards this direction, when we are pulling it out it pushes this side. So, that gives us the condition of passive earth pressure; that is why we said that this is nothing but a theory of passive earth pressure. Now why it is negative wall friction angle, because in this case what happens, your wall moves up and the surrounding soil this one moves down. So, that is the relative movement between these two zones. So, in this case of passive earth pressure, whatever is happening here on this one, this face; that is acting on the other direction; that is it will be a negative passive earth pressure not a positive one, which is occurring for bearing capacity theory for shallow foundation. So, that is why it is called negative wall friction angle case. So, it can be either planer rupture surface or curvilinear rupture surfaces, depending on what type of passive earth pressure theory you are going to use.

Now with this basic concept of ground anchor and its application, using passive earth pressure theory concept. Now this problem becomes more complex under the seismic condition, and these are the pictures which shows the practical applications of this ground anchors as I said, we put the ground anchors here, in this foundation support of transmission towers always at four corners, so that it does not lift up like this. But what are the causes of this uplift force or tensile force or pullout load, like there will be always huge lateral load, like wind load, seismic load etcetera, because of that it will try to pull out. So, in seismic condition also there will be additional pullout load, which will be acting on the super structure, which finally you need to take care, in the ground anchor design. So, these pictures shows you can have still helical anchor or concrete anchors or various types of materials can be used as anchor plate.

Even for the submerged pipeline, those pipelines also are always put with anchor support, at different intervals of the length of the pipeline, otherwise what will happen. When there is an uplift force, the pipe line will come out from the embedment, and it will open up. So, nobody will like to see, suppose some waste water line which is going underground. If the proper anchoring system ground anchors are not provided, at some point of uplift pressure, if it is more than the surcharge pressure, at which depth the pipeline has been constructed it can come out. So, that is why to provide the stability, to maintain the stability of the entire system, this ground anchors are very important. Another application as we know, for the sheet pile design, for anchored sheet pile; that is sheet pile to reduce its excessive movement, excessive lateral movement, we provide the plate anchors like this. So, that the sheet pile, is tied in such a way, that it does not deflect excessively.

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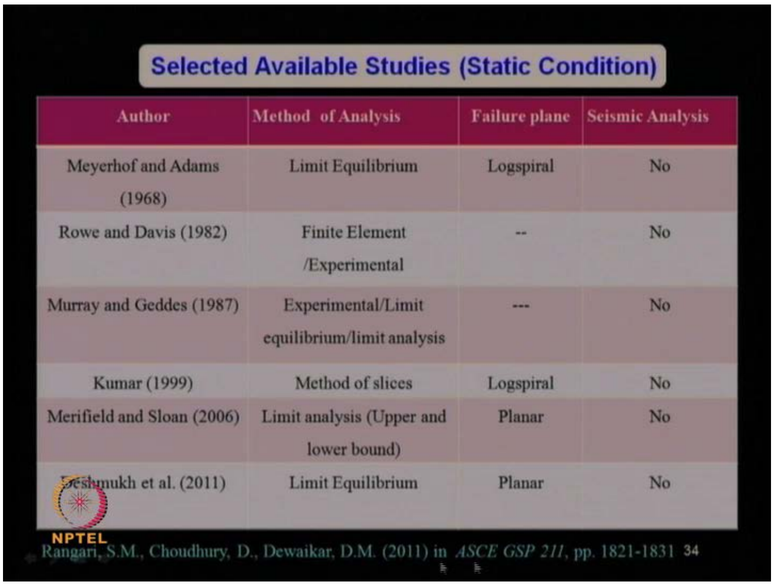
Selected Available Studies (Static Condition)

Author	Method of Analysis	Failure plane	Seismic Analysis
Meyerhof and Adams (1968)	Limit Equilibrium	Logspiral	No
Rowe and Davis (1982)	Finite Element /Experimental	--	No
Murray and Geddes (1987)	Experimental/Limit equilibrium/limit analysis	---	No
Kumar (1999)	Method of slices	Logspiral	No
Merifield and Sloan (2006)	Limit analysis (Upper and lower bound)	Planar	No
Deshmukh et al. (2011)	Limit Equilibrium	Planar	No

Rangari, S.M., Choudhury, D., Dewaikar, D.M. (2011) in *ASCE GSP 211*, pp. 1821-1831 34

So now, let us see, what are the available literatures on this anchor? Actually on ground anchors, there are extensive research work was carried out by several researchers, under static loading condition. So, first I am addressing here static loading conditions. In this slide we are mentioning only a very few of them, there are many others which are not listed over here. This is just to give an overall idea that what are the major research going on, for horizontal strip anchor, horizontal shallow strip anchors. So, the one of the pioneering work in this ground anchor area, was done by Meyerhof and Adams, in 1968, using the simple limit equilibrium method. And they considered the log spiral failure surface, but no seismic analysis was done, as I said this is the static loading conditions. So, in this column of seismic analysis, for all of them it is no, because it is a static loading cases I am referring to. Another classical work was done by Prof. Rowe and

Prof. Davis in 1982, using finite element as well as the experimental values, these are available.

Then Murray and Geddes in 1987, they also obtained experimental results, as well as they used limit equilibrium method, also they used limit analysis method. Kumar in 1999 used method of slices, using log spiral failure surface to obtain the uplift capacity of anchors under static condition. Then Merrifield and Sloan in 2006 used limit analysis, both upper and lower bound, using a planer rupture surface or planer failure surface for anchor. And recently Deshmukh et al, he is another PhD student who completed his PhD at IIT Bombay, under the supervision of Prof. Dewaikar and myself. So, he worked using limit equilibrium concept, for the uplift capacity of anchor using planner failure surface, under static condition only. So, all this literature review and the state of the art concept for a ground anchor. You can find in this research paper, Rangari Choudhury and Dewaikar 2011 in ASCE's Geotechnical special publication number 2 1 1. These are the page numbers of the paper.

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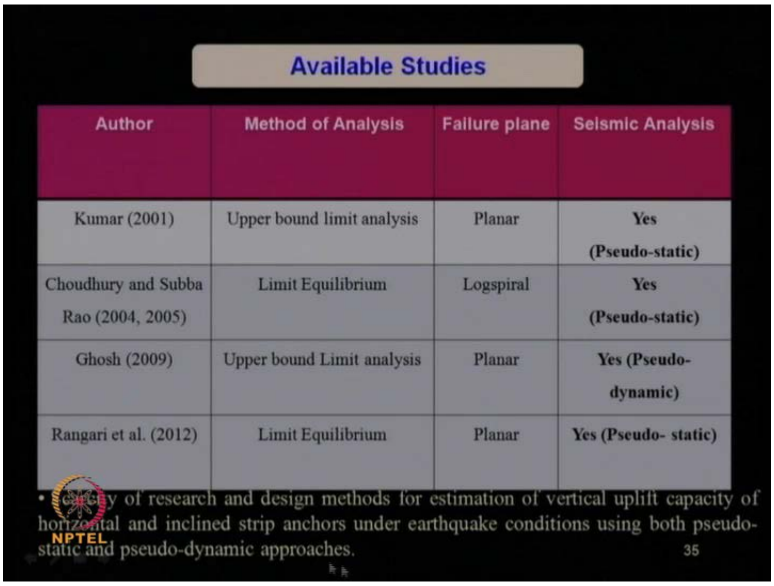

Now coming to the seismic analysis; that is under earthquake condition who are the researchers, who did research for the estimation of vertical uplift capacity of horizontal and inclined strip anchors under earthquake condition. Like Kumar in 2001; that was one of the pioneering work to obtain the uplift capacity of ground anchors under earthquake condition, using pseudo-static approach. So, he used upper bound limit analysis, using a planer failure surface, and pseudo-static method was used for seismic analysis. Then myself during my PhD I worked under the supervision of Prof. K S Subba Rao at IISC Bangalore. So, we had publications Choudhury and Subba Rao 2004 and 2005. This 2004 is in the journal Geotechnical and Geological Engineering of Springer, and 2005 is the Canadian Geotechnical journal. We used limit equilibrium method one is for horizontal strip anchor; another is for inclined strip anchor.

We used log spiral failure surface and also for seismic forces, we used the concept of pseudo-static. Then Ghosh in 2009 he used upper bound limit analysis using planer rupture surface, but he proposed pseudo-dynamic method to get applied for estimating this uplift capacity of anchor. So, he used our proposed pseudo-dynamic model, as I have discussed earlier, for the uplift capacity determination of anchors also. So, as I am currently mentioning, the PhD thesis work of Rangari. So, Rangari et al 2012; our one paper, we discussed about limit equilibrium method using planer rupture surface and here we used pseudo-static method, but later on we used pseudo-dynamic also, which I am showing in couple of next slides. So, you can see from these researchers results, that there is a scarcity of research and design method for estimation of vertical uplift capacity of both horizontal and inclined strip anchors, under earthquake conditions, using both pseudo-static approach as well as pseudo-dynamic approach.

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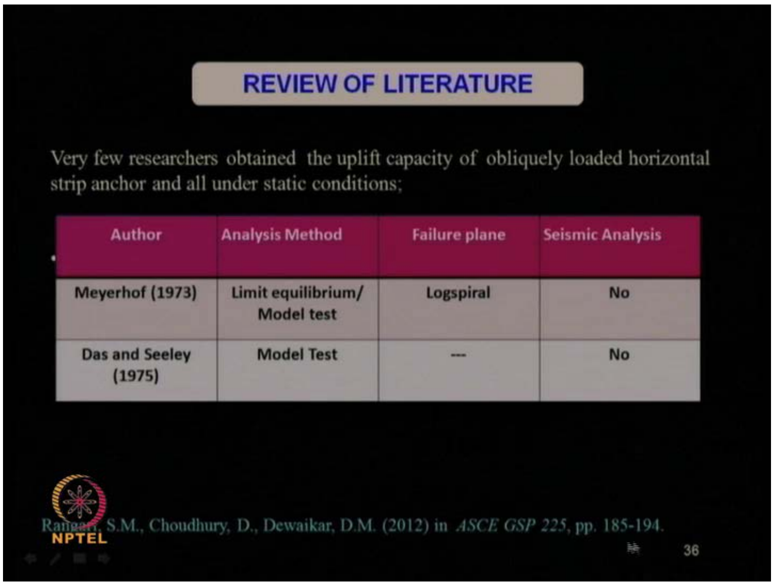

Author	Analysis Method	Failure plane	Seismic Analysis
Meyerhof (1973)	Limit equilibrium/ Model test	Logspiral	No
Das and Seeley (1975)	Model Test	---	No

And if we talk about obliquely loaded horizontal strip anchors, or inclined strip anchors also; that is horizontal strip anchor is a very common application, but inclined strip anchors also are used at many places, as I have already mentioned at various technical sites, we need to probably provide the foundation at little inclined position, or the anchors in little inclined position. So, inclined anchor plates are also very much in used in practice. So, for those inclined plate anchors, or the horizontal plate anchors, many a time what happens, the pullout load or the tie rod will not be vertical, it will be somewhat inclined. So, in that case it is subjected in oblique load or inclined load. It depends on your, the direction of the force which is coming, on the foundation system.

So, this obliquely loaded horizontal strip anchor, even on the static condition also, there is very few researchers who worked in this area, though there is lot of application of this problem as I have already shown. Like Meyerhof again in 1973, he proposed Limit Equilibrium method, as well as he has shown the model test results. For Limit Equilibrium method he used log spiral failure mechanism, and it was static case, so that is why seismic analysis is no. Then Das and Seeley in 1975 used the Model test for this obliquely loaded horizontal strip anchor, and it is only for static loading condition. So, seismic analysis is no. So, the state of the art literature review, you will get in this paper in detail Rangari Choudhury and Diwaikar 2012, in ASCE's geotechnical special publication number 225. These are the page numbers.

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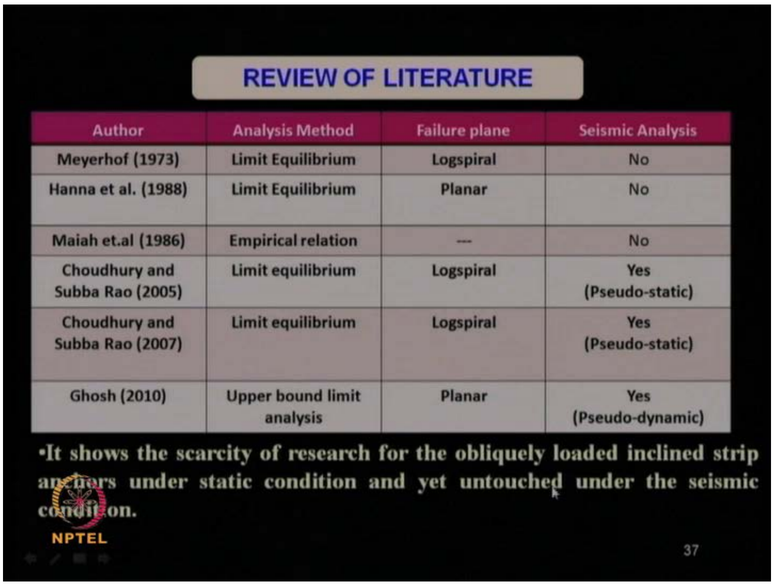

Author	Analysis Method	Failure plane	Seismic Analysis
Meyerhof (1973)	Limit Equilibrium	Logspiral	No
Hanna et al. (1988)	Limit Equilibrium	Planar	No
Maiah et.al (1986)	Empirical relation	---	No
Choudhury and Subba Rao (2005)	Limit equilibrium	Logspiral	Yes (Pseudo-static)
Choudhury and Subba Rao (2007)	Limit equilibrium	Logspiral	Yes (Pseudo-static)
Ghosh (2010)	Upper bound limit analysis	Planar	Yes (Pseudo-dynamic)

Coming to some more researcher's results, in this area of obliquely loaded and the inclined strip anchor. Like for inclined strip anchor Meyerhof in 1973 Limit Equilibrium method as I said, log spiral failure mechanism, no seismic analysis. Hanna et al in 1988, they also used Limit Equilibrium method using planer failure surface, and no seismic analysis. Maiah et al in 1986 use Empirical relation, it is also for static analysis. Whereas Choudhury and Subba Rao, we did the research for inclined strip anchors as I have said, using limit equilibrium method, using log spiral failure surface, and it was done for seismic analysis also using pseudo-static approach. Then Choudhury and Subba Rao 2007, again limit equilibrium approach log spiral, it was also pseudo-static approach. And Ghosh in 2010, Prof. Priyanka Ghosh of IIT Kanpur. He worked in this area of upper bound limit analysis, using planer rupture surface, he used pseudo-dynamic results. So, it again shows the scarcity of the research, for obliquely loaded and inclined strip anchors under both static condition, and it is not yet touched in the seismic condition. Because seismic condition obliquely loaded, nobody has done. These are the inclined strip anchors, but not inclined strip anchors with obliquely loaded, inclined strip anchor with vertical load.

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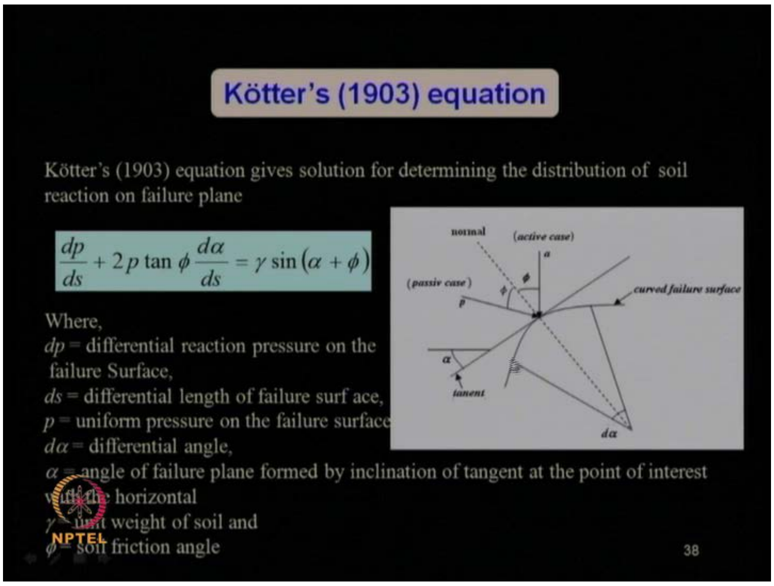

Now what Dr. Sunil Rangari's PhD thesis comprises of. He used the basic Kotter's equation; Kotter's equation, it was proposed in 1903. This mathematical expression for defining the distribution of reaction, on any curvilinear failure surfaces is shown like this. So, this is the basic Kotter's equation. You can see over here, this is curved failure surface, this is the tangent at any point, normal to this curvilinear failure surface. If this horizontal angle of this tangent with this horizontal is alpha, this infinitesimal small angle is d alpha. You have for passive case this direction, for active case this direction. So, the basic equation governing equation for Kotter's equation which was proposed by Kotter in 1903, is given by this d p by d s plus 2 p tan of phi d alpha by d s, equals to gamma times, sine of alpha plus phi, where this d p is differential reaction pressure on the failure surface, d s is the differential length of the failure surface, p is the uniform pressure on the failure surface; d alpha is the differential angle over here. Alpha is the angle of failure plan, formed by inclination of the tangent at the point of interest with respect to the horizontal as I said here. Gamma is the unit weight of the material. So, in our case it is the soil and phi is a soil friction angle. So, this equation is valid for curvilinear failure surface. So, if somebody wants to use the planer failure surface, it needs to be modified.

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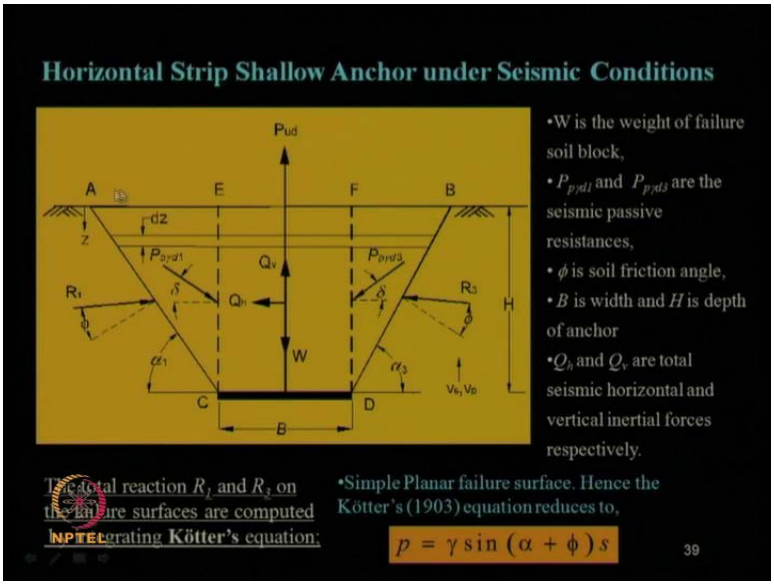

So, simple planer failure surface, the equation that reduces to or simplified to this value, because in that case this term will not be there, because there is no variation of this d alpha by d s. So, you will get this d p by d s, equals to gamma time sine of alpha plus phi. So, p equals to you can integrate gamma time sine, alpha plus phi times S. So, this is for simpler failure surface. So, what Sunil Rangari's thesis comprises of. He first carried out the research on the area of horizontal strip anchor, horizontal strip shallow anchor under seismic conditions. So, this is the horizontal anchor, subjected to vertical uplift load. See all this conditions, and width of the anchor plate is B, depth of the embedment of anchor plate is H and this is strip anchor, so length of this anchor is much more than this width. So, this P ud that is the uplift load for this anchor, under dynamic condition needs to be obtained. He considered the planer rupture surface like this, simplest planer rupture surface, but used the Kotter's equation. So, when you are using Kotter's equation, the advantage is you are getting the soil reaction on this site, which is unknown earlier, as I have mentioned to you in one the previous lecture.

So, from this equation you can get these values of this total reactions R1 on this site, and R3 on this site, is it ok, based on the planer rupture surface. Now these two zones are different, because at one instant of acceleration or direction, obviously it will be, one side will be larger zone, another side will be a smaller zone of failure. It will not be symmetrically equal, or it should not be symmetric, as the case is under static conditions. So, that is why the static loading condition solution is, comparatively simpler than the seismic loading condition. And these are the seismic inertia force in vertical direction; in horizontal direction and this is the weight of this central block this E C D F. And these are the imaginary retaining wall C E and D F, on which we are considering this seismic passive earth pressure, which are acting at angle delta. And that passive earth pressure has to be obtained for this failure zone D B F, and this passive earth pressure has to be obtained for this failure zone C E A.

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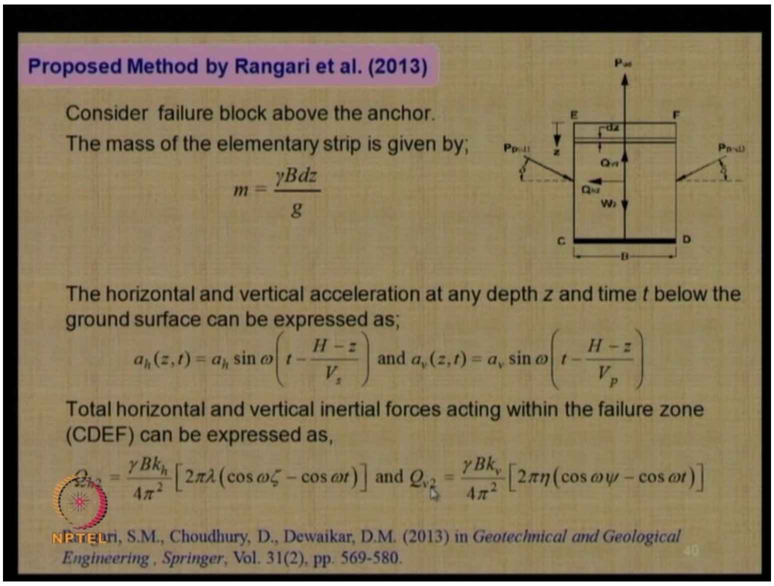

So now, if we consider this central portion, this detail of this proposed method is available in the journal paper by Rangari et al 2013. You can see the details of this paper Rangari Choudhury and Diwaikar 2013, in the journal Geotechnical and Geological Engineering, Springer publication. This is volume issue number and page numbers. So, what has been done, once you get the passive earth pressure, then these are the forces which are acting on the central block, and you are considering this infinitesimal small element. Now, if you are applying the pseudo-dynamic approach as we have applied here. So, these are the horizontal and vertical seismic acceleration, as we have already discussed for pseudo-dynamic case; that will give you the seismic inertia forces in this zone, which you can obtain by integrating these inertia forces over the entire depth of this anchor plate. So, that is how you get these values of q h 2 and q v 2; that is the horizontal inertia force, seismic inertia force and vertical seismic inertia force.

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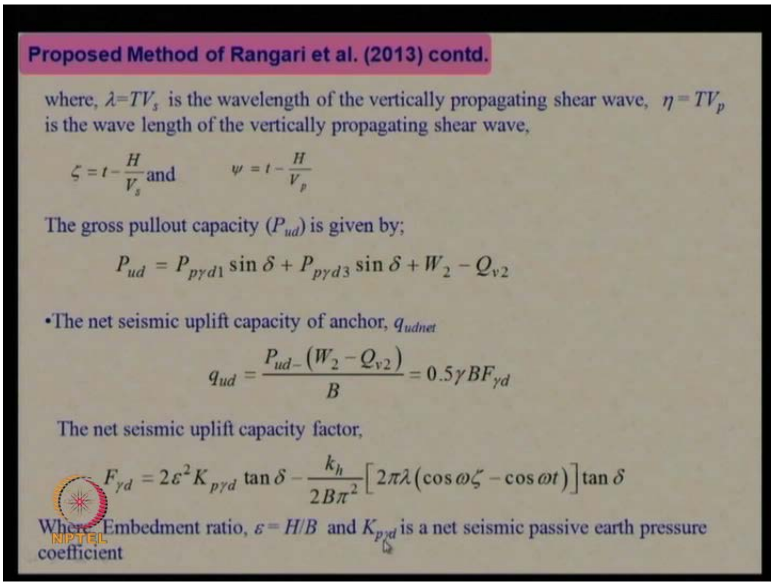

Now, once you get that, then you consider the equilibrium of all the forces involved, both in horizontal direction as well as vertical direction. So, satisfying the equilibrium what you will finally get, you will get the expression in terms of P ud, which you want to find out. That is gross pullout capacity under dynamic condition, and we express them in terms of net seismic uplift capacity q ud net in this form, which is given in the common form of half gamma BF gamma d, what is F gamma d. F gamma d is called anchor uplift capacity factor. Like in case of foundation, shallow foundation we have seen n gamma d, which is called bearing capacity factor. For anchor it is called s gamma or F gamma d, as proposed by Meyerhof also, it is known as anchor uplift capacity factor. So, this is the closed form solution of the anchor uplift capacity factor, in terms of embedment depth, in terms of passive earth pressure coefficient, which is acting on the different site of this failure plane.

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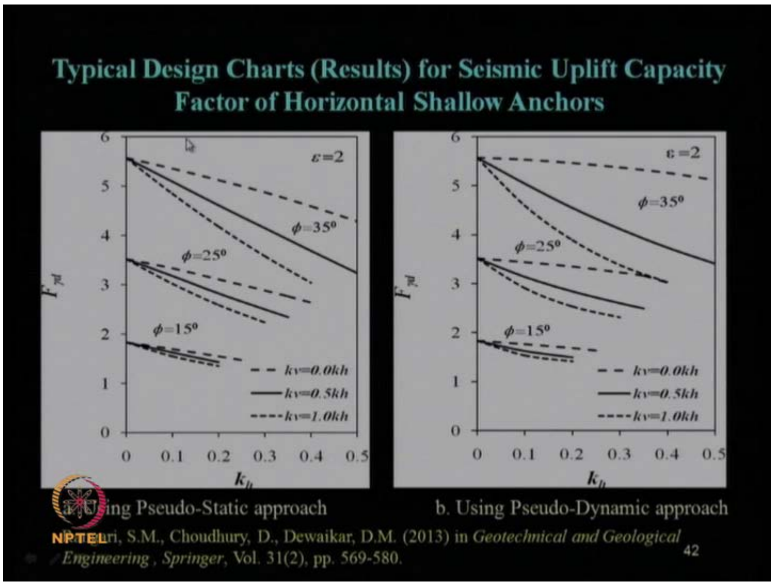

So, finally, the typical design charts are proposed like this, which will help the designers to design any anchor plate in this seismically active region. Suppose if you have k h value for a zone 0.2 g, you have a particular value of k v, say half of k h, and phi value let us say 35 degree. You can go here depending on your embedment depth, remember this value, embedment depth of two you will get some value of F gamma d, which will help you to get the net ultimate and gross ultimate values of this, net ultimate seismic uplift capacity factor q ud net, similarly for other cases also. So, this is using pseudo-static approach, and similarly using pseudo-dynamic approach this design charts have been proposed in this paper, which one can easily get.

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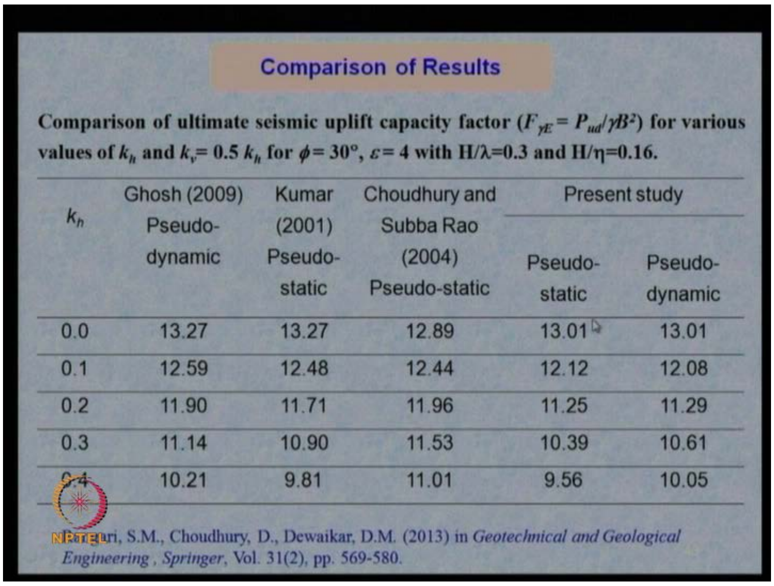

Now what is the validation of this results, you can see the results have been compared and with previous researchers results, using the ultimate seismic uplift capacity factor; this F gamma E, which is expressed as p ud by gamma B square in that non dimensional form. So, these are non dimensional factors, as we have seen. So, for various input values, so these are the different input values you can see over here. For different values of KH Ghosh pseudo-dynamic approach, these are the results of this F gamma E. Kumar’s result using pseudo-static approach, these are the values. Choudhury and Subba Rao’s pseudo-static approach 2004, these are the values.

Whereas the present study give the pseudo-static values like this and pseudo-dynamic values like this. So, you can see the present study using this Kotter’s equation, gives the minimum value of this seismic uplift capacity factor F gamma E, what does it mean. It automatically shows it is showing the most critical design, or the better design, as for as the ground anchors under seismic condition is concerned. Why it is so, because the advantage as I said in Kotter’s equation you are getting your satisfying the condition of soil reaction at the failure surface, and your considering that at each and every point of the curve. So, that is the important addition compared to the other researches what they have done, and moreover pseudo-dynamic case.

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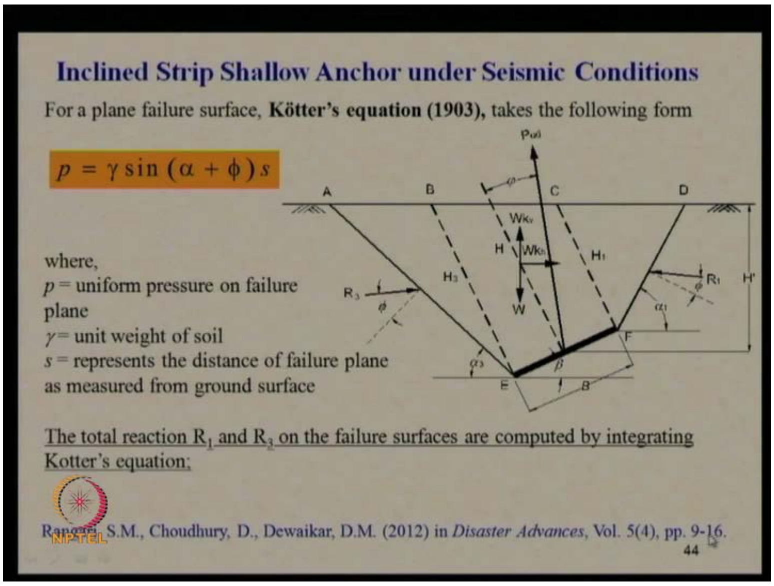

Now, coming to the application for the case of inclined strip anchors. So, if the anchor plate is like this inclined, say the inclination angle of the anchor plate is beta with respect to the horizontal. These are the soil reactions, which your getter again from the Kotter's equation, and these are the various seismic. This is the weight of the central block and seismic inertia forces in vertical and horizontal direction. This is the uplift capacity. Now remember this inclined anchor, is again is loaded obliquely. Load is not perpendicular to the plan of the anchor plate, can you see that. So, this is, this shy is the angle of inclination of load applied. It is quite possible, that you have used the anchor plate inclined like this, and your load of action or line of action of load can be inclined to the vertical on that plane. So, for that complicated condition, how the reactions and the uplift capacity have to be determined, that has been analyzed. The details you can find in this journal paper Ragari, Choudhury and Diwaikar 2012, in the journal Disaster Advances. This is the volume number and page issue number, and page number.

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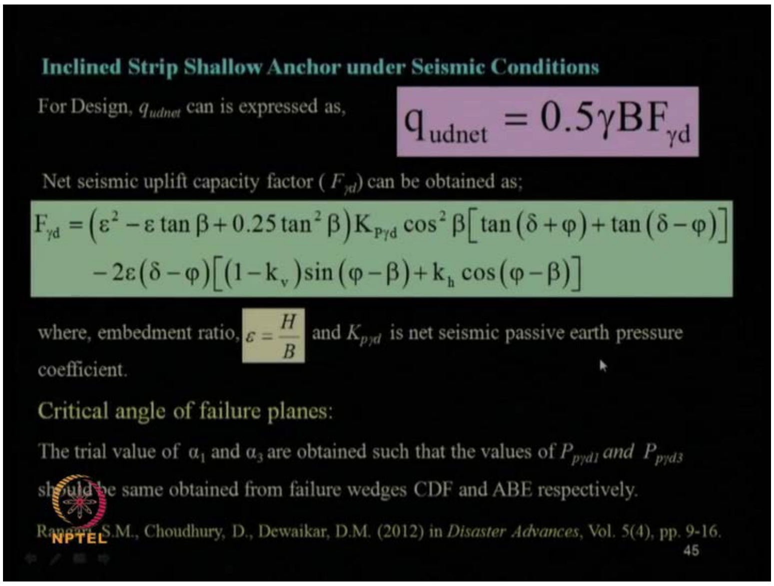

So, here also, the q udnet is expressed in this form, half gamma B and F gamma d. F gamma d is non dimensional uplift capacity factor. Or the designed uplift capacity factor under seismic condition, and the closed form equation of that F gamma d is given this expression. You can see these are function of this embedment ratio, anchor inclination k p gamma d, which is again in terms of function of phi value, and the seismicity parameters k h k v. Also in addition to that these are functions of k h k v, delta value and the inclination of your load of action on the plane; that is shy. So, the critical angel of failure, critical angle of failure means. The angle of failure at which this alpha 1 and alpha 3, will give you the minimum value of this p ud that is nothing but your critical angle, that needs to be determined using, optimization technique; that is you have to use the optimization method to find out the minimum value of this failure planes, so that you get the minimum p ud value.

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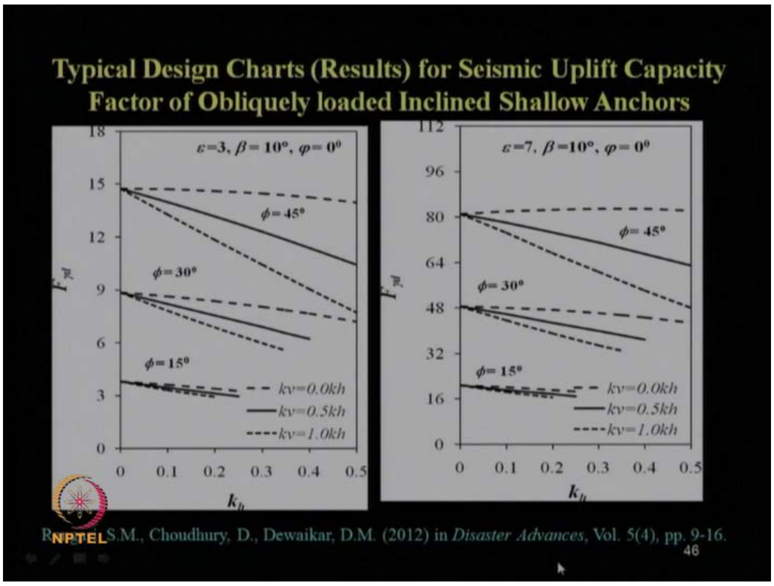

Again, here also, the design charts have been proposed for practical use by designers, for a particular value of k h and k v, for a known value of phi, one can get the F gamma d value, for the given value of embedment depth, anchor inclination and load inclination. Load inclination zero means, the load is acting exactly perpendicular on the inclined plan of the anchor.

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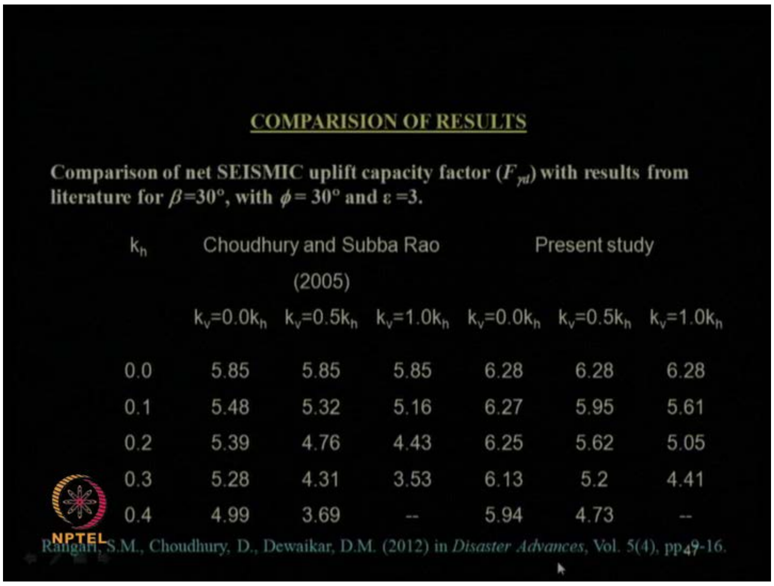

So, for different other angles also results are available in this journal paper. Here also the comparison of results have been made with, other inclined researchers result. There is only few research available like Choudhury and Subba Rao research 2005 is for inclined anchor, that has been compared here with the present study results.

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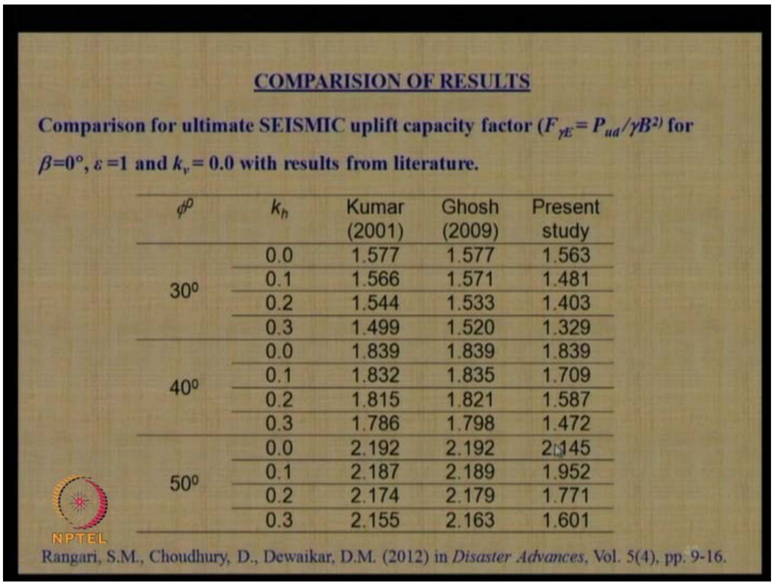

ϕ^0	k_h	Kumar (2001)	Ghosh (2009)	Present study
30⁰	0.0	1.577	1.577	1.563
	0.1	1.566	1.571	1.481
	0.2	1.544	1.533	1.403
	0.3	1.499	1.520	1.329
40⁰	0.0	1.839	1.839	1.839
	0.1	1.832	1.835	1.709
	0.2	1.815	1.821	1.587
	0.3	1.786	1.798	1.472
50⁰	0.0	2.192	2.192	2.145
	0.1	2.187	2.189	1.952
	0.2	2.174	2.179	1.771
	0.3	2.155	2.163	1.601

And the results with Kumar and Ghosh with the present study, for the cases of horizontal anchor, beta equals to 0 degree is nothing but inclination is 0 degree means horizontal anchor; that has been compared over here. So, with this, we have come to the end of today's lecture, we will continue further in our next lecture.