Geotechnical Earthquake Engineering

Prof. Deepankar Choudhury

Department of Civil Engineering

Indian Institute of Technology, Bombay

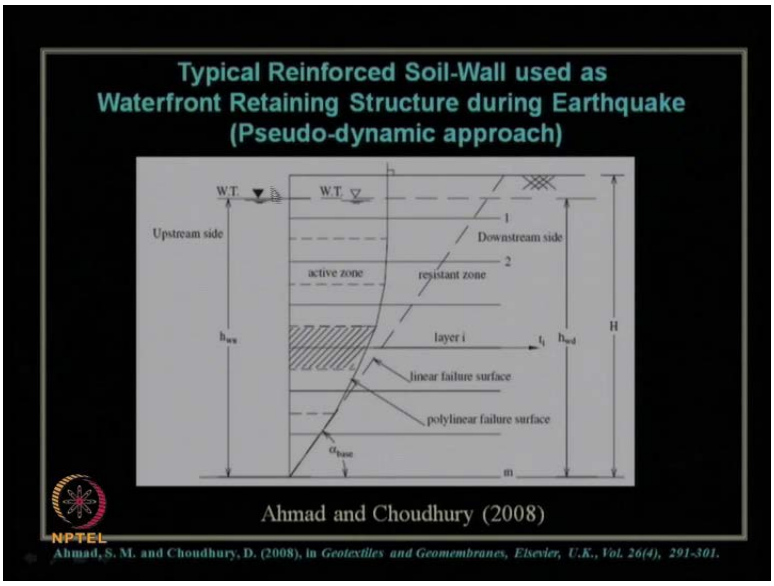
Module - 9

Lecture - 39

Seismic analysis and design of various geotechnical structures (Contd…)

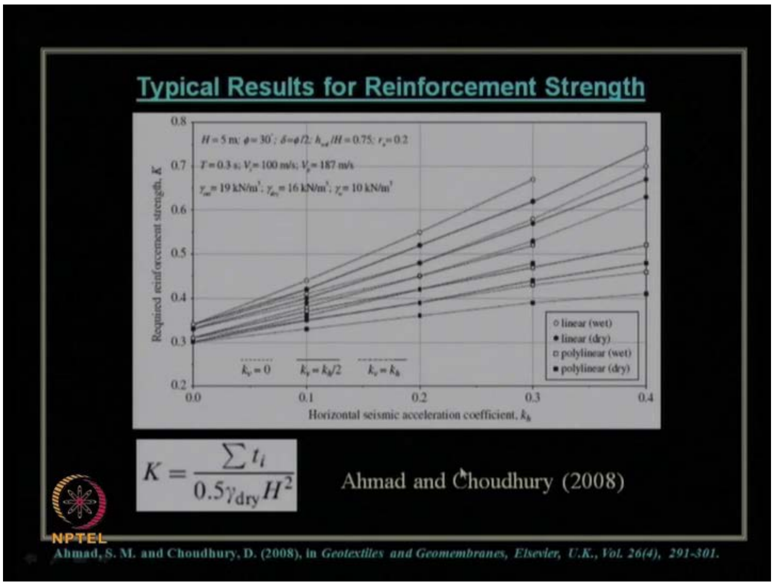
Let us start our today's lecture, for this NPTEL video course on Geotechnical Earthquake Engineering. Currently, we are going through our module number nine, which is on seismic analysis and design of various geotechnical structures. A quick recap, what we have learnt in our previous lecture.

(Refer Slide Time: 00:45)



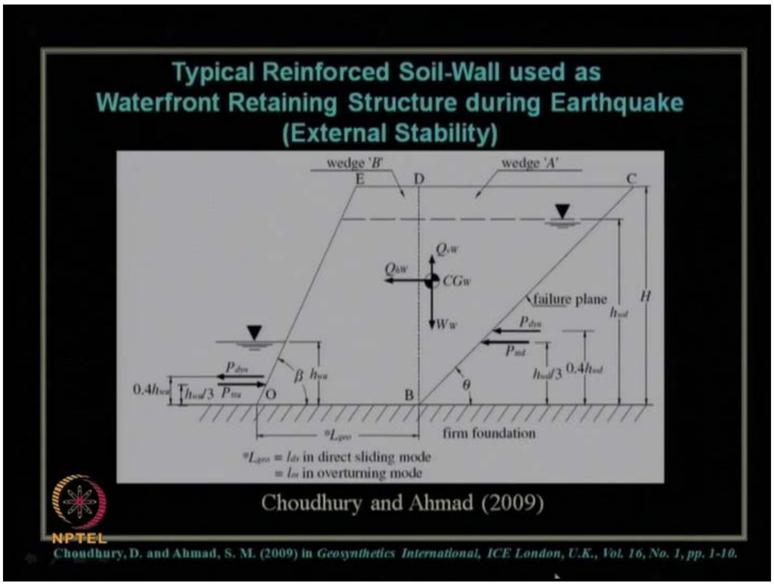
We studied the seismic design of waterfront reinforced soil wall; that the analysis of reinforce soil wall in the water front, by using Pseudo-dynamic approach, both in terms of internal stability as well as external stability. The internal stability this is the paper, by Ahmad and Choudhury, appeared in journal Geo-textile and Geo-membranes Elsevier publication 2008.

(Refer Slide Time: 01:09)



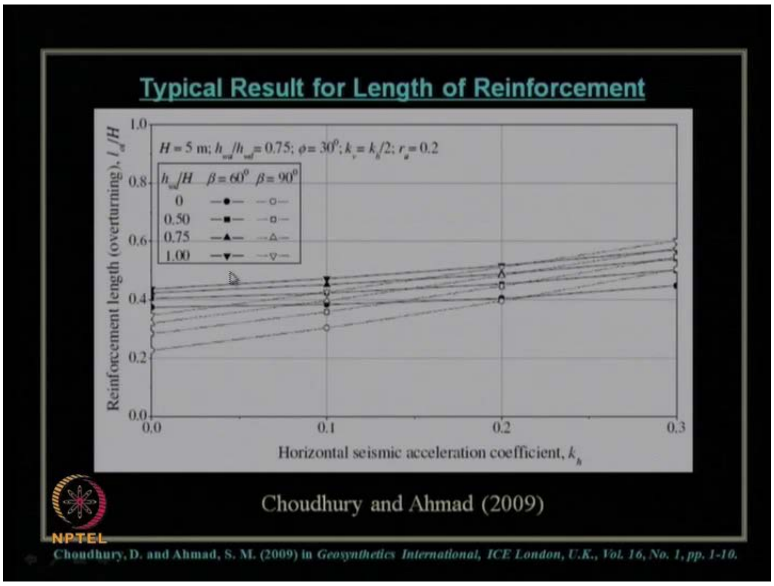
We mentioned how to find out the required reinforcement strength, to be provided for internal stability.

(Refer Slide Time: 01:17)



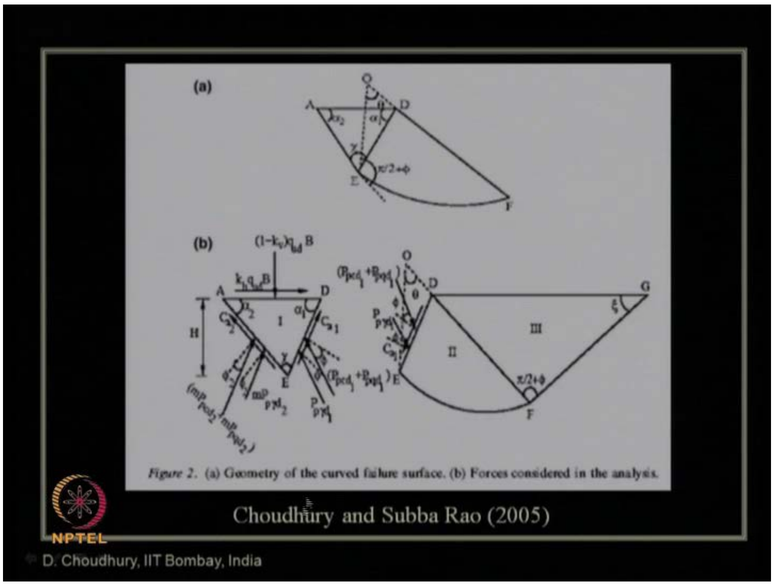
Also based on the external stability of that reinforced soil wall in the waterfront, which is available in the publication of Choudhury and Ahmad, in the journal in Geo-synthetics International IC London publication.

(Refer Slide Time: 01:31)



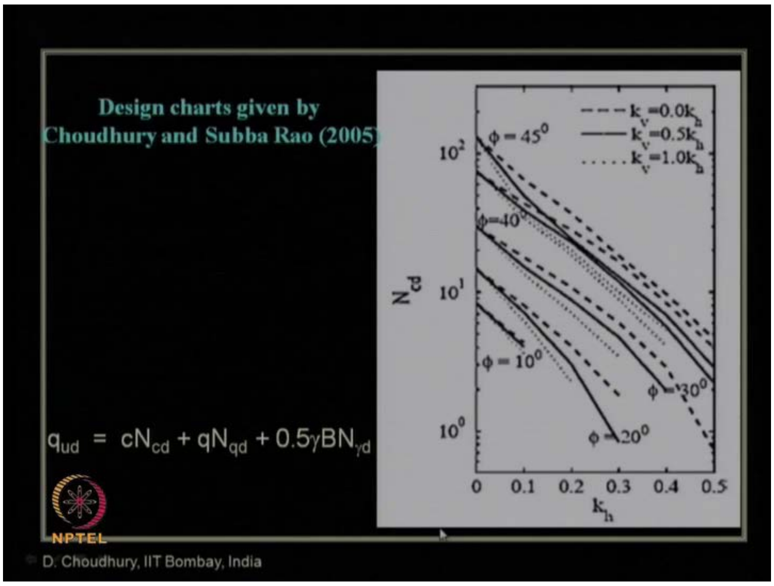
We found out how much reinforcement length is required, in terms of overturning mode of failure, in terms of sliding mode of failure. And then among these three length; that is one is pullout of the reinforcement, another is sliding, another is over turning. Based on all these three, you have to provide the maximum length of the reinforcement, for stability of such reinforce soil wall in the waterfront. Then in our previous lecture, we also discussed about seismic design of shallow footings.

(Refer Slide Time: 02:06)



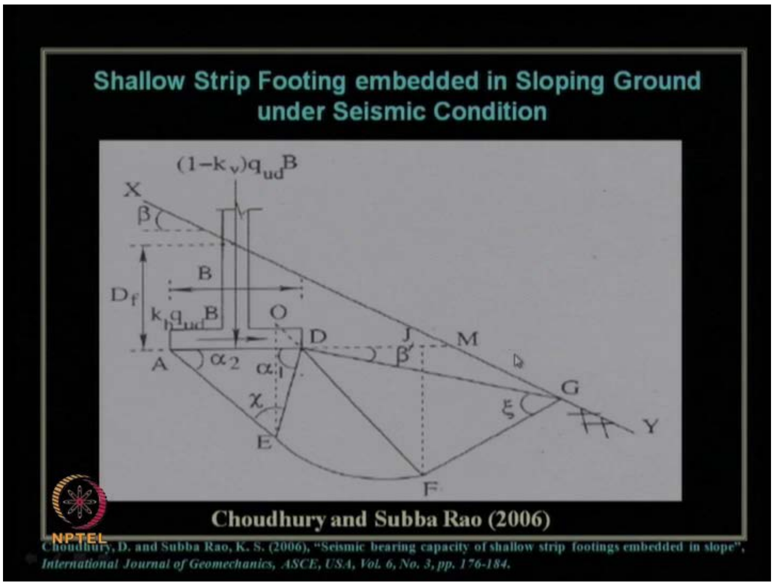
For that, first we have started with the pseudo-static analysis of shallow strip footing, as proposed by Choudhury and Subba Rao in 2005.

(Refer Slide Time: 02:15)



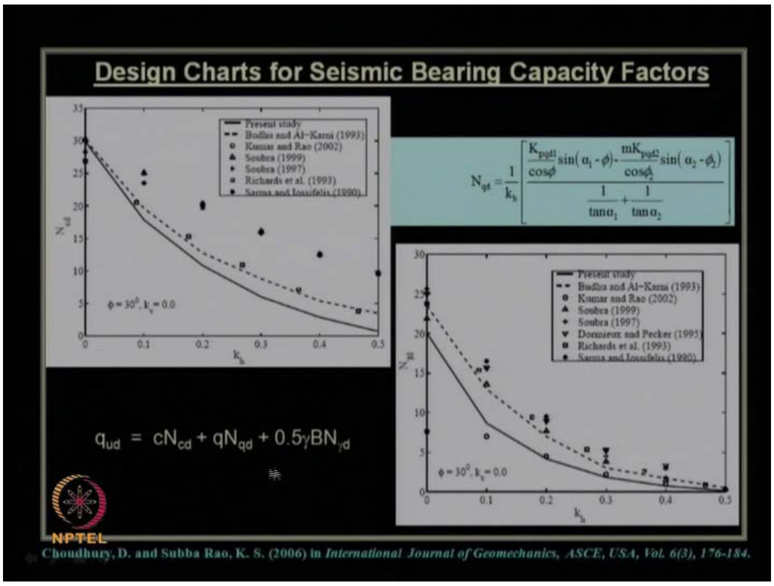
And the design charts, under the dynamic condition or seismic condition, is proposed like this is the bearing capacity factor in terms of cohesion; that is N c d. And this is the equation by using which, one can estimate what is the seismic baring capacity, q u d is seismic bearing capacity equals to c N c d; that is the cohesion component, q N q d surcharge component, plus half gamma BN gamma d. So, it is exactly same as the Terzaghi’s equation of bearing capacity, but the modified in terms of the seismicity is concerned, in terms of pseudo-static seismic acceleration. So, in this N c d, N q d and N gamma d are seismic bearing capacity factors, using Pseudo-static approach. We can see as the seismicity increases; that is K h value increases and K v value increases, there is a significant decrease in this value of this bearing capacity, which needs to be considered for design of any shallow footing in a seismically active region.

(Refer Slide Time: 03:27)



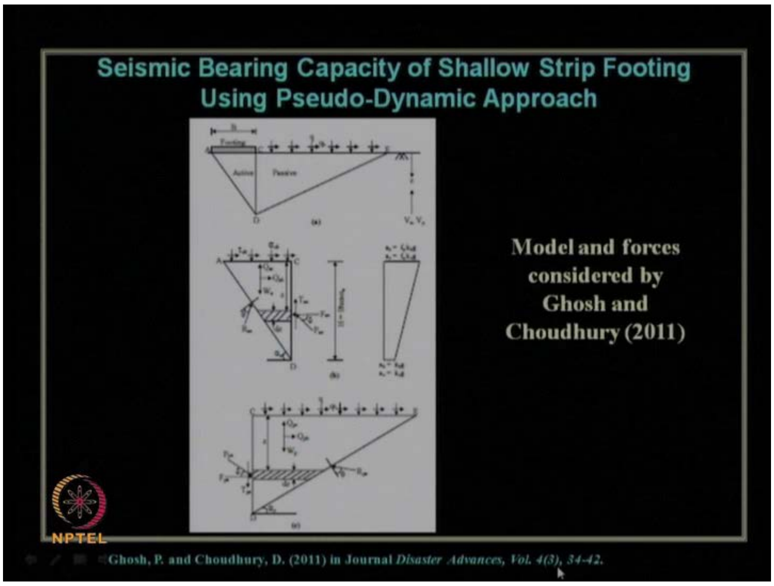
These are the design chart for N q d, these are design charts for N gamma d. Again we discussed about the design concept of shallow strip footing, embedded in sloping ground; that is when we are embedding it in the sloping ground, many a cases like in hilly terrain hilly region, we have shallow strip footing like this, which needs to be designed and constructed in the sloping ground like this. So, how to design that, the details are given in this journal paper by Choudhury and Subba Rao 2006, in international journal of Geo-mechanics ASCE 2006 issue.

(Refer Slide Time: 04:04)



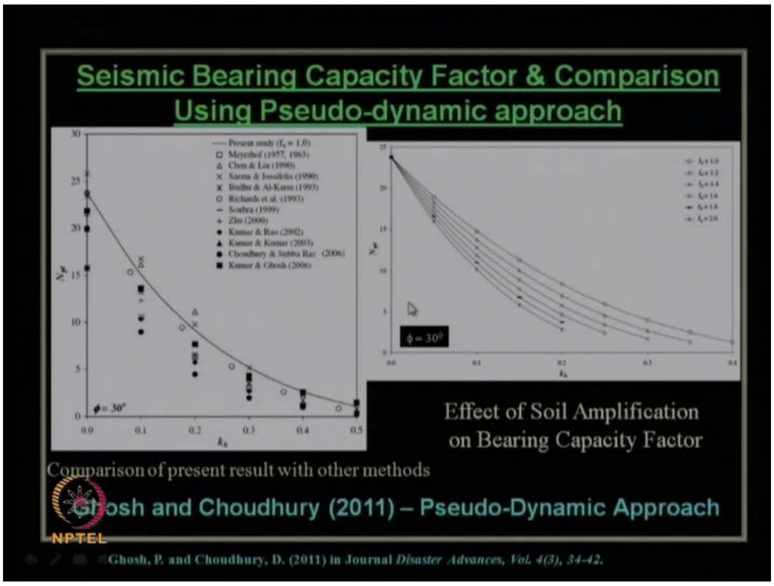
So, here also the design charts have been proposed in terms of N c d, N q d and N gamma d, including their closed form solutions, and the seismic bearing capacity can be estimated using this proposed equation.

(Refer Slide Time: 04:19)



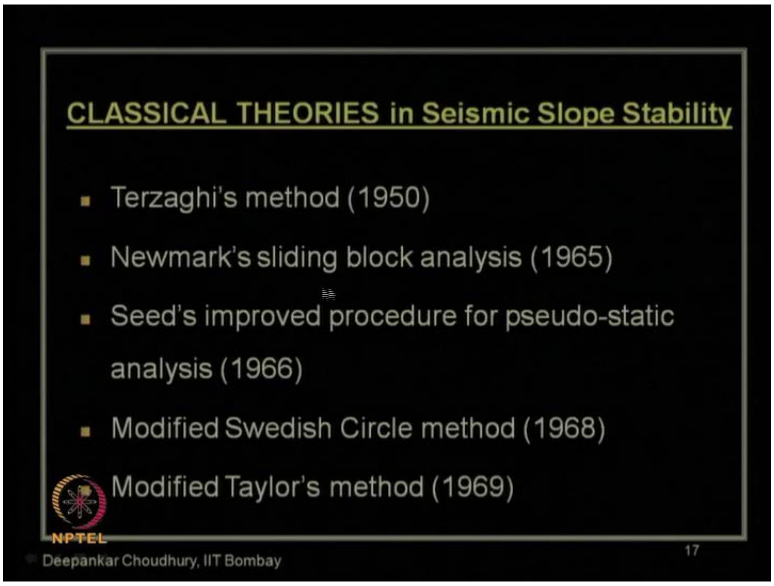
Next in our previous lecture, we also discussed about how to estimate the seismic bearing capacity of shallow footing, using the Pseudo-dynamic approach, which is proposed first time by Gosh and Choudhury in 2011, which is available in the journal disaster advances. This is the volume and page number. Here only two wedge failure mechanism was considered, along with the amplification factor.

(Refer Slide Time: 04:47)



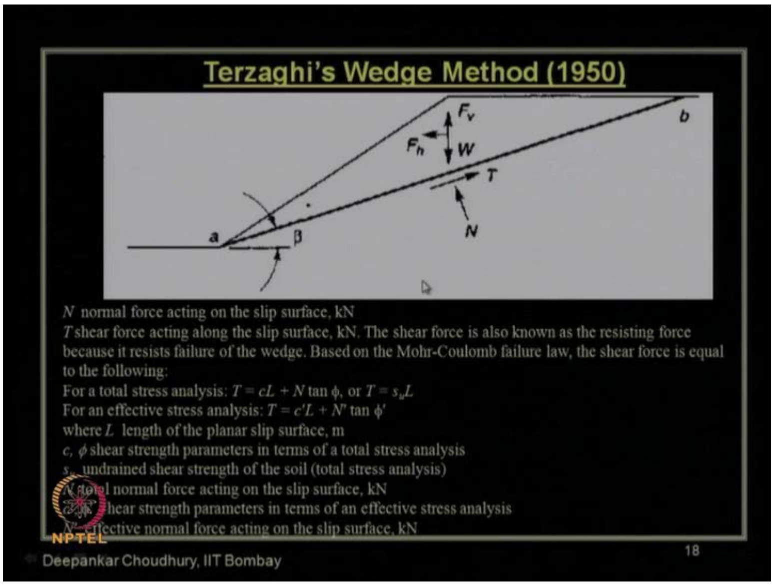
And we have seen that, in Pseudo-dynamic approach we can consider the effect of soil amplification, and that reduces, or decreases the seismic bearing capacity factors; significantly, which is not possible to consider in the conventional pseudo-static approach. So, that way Pseudo-dynamic approach is much better than pseudo-static approach, whenever this soil amplification and other dynamic parameters are coming into picture. Next we started in our previous lecture, with the sub topic on seismic stability of finite soil slope.

(Refer Slide Time: 05:27)



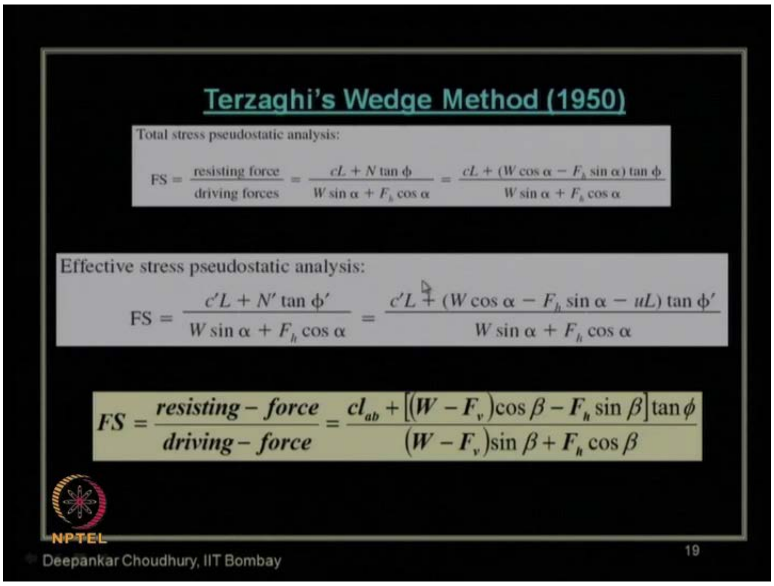
We discussed first about the classical theories in seismic soil slope stability analysis, using conventional pseudo-static approach, which was started with Tarzagi's approach in 1950, then followed by Newmark's sliding block method 1965 and so on.

(Refer Slide Time: 05:44)



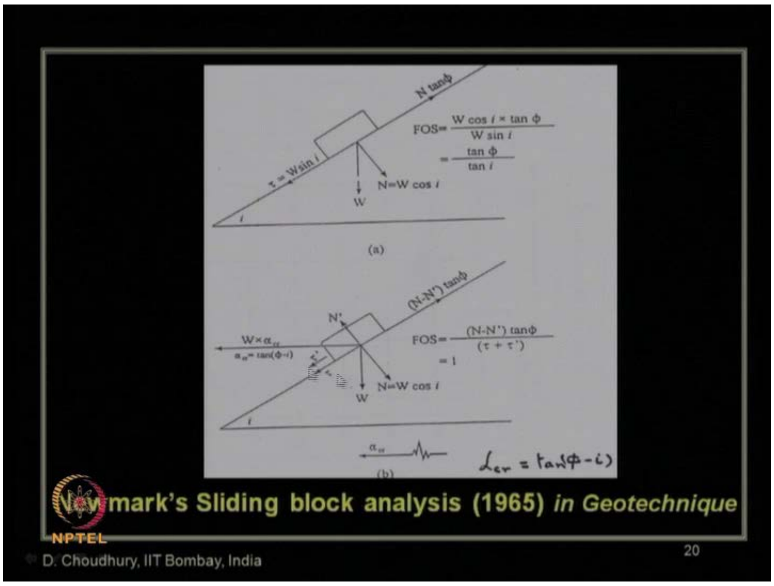
So, this is Tarzagi's wedge method for the slope stability. We had considered in our previous lecture; that if this is the weight of the failure soil mass. These are the seismic inertia forces in horizontal and vertical direction.

(Refer Slide Time: 06:00)



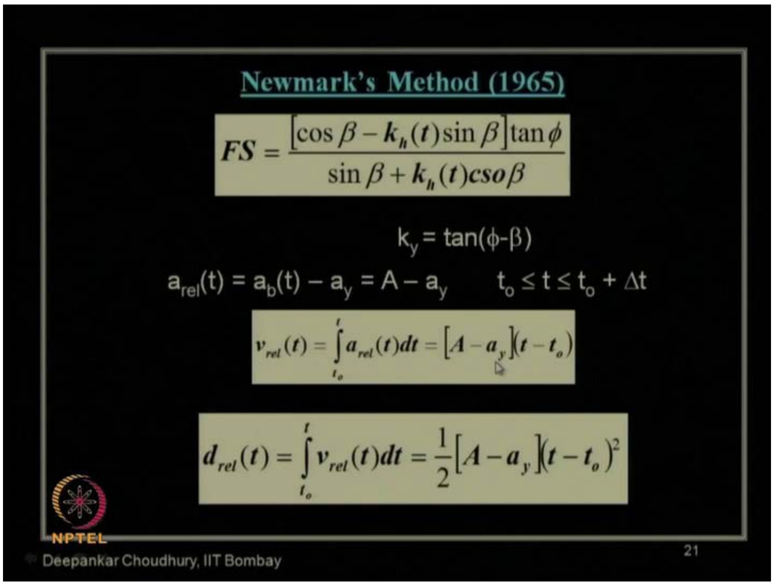
Then as per the Tarzagi's analysis, we can get the factor of safety expression, which is nothing but ratio of resisting force by driving force, as expressed in this format.

(Refer Slide Time: 06:20)



And it is recommended that for stability, generally factor of safety needs to be more than 1.15; whereas, Newmark's sliding block method; which is another advance method than Tarzagis analysis, which appeared in the journal geotechnical in 1965. It is one of the pioneers in work, because it extended the basic concept of the blocks sliding over a sloping ground, in the static case to the seismic event, considering this seismic inertia or additional disturbing force like this.

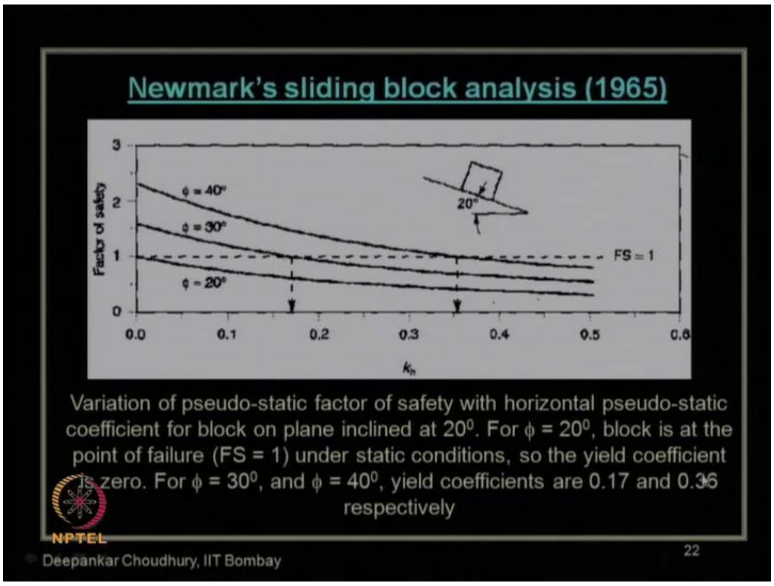
(Refer Slide Time: 06:48)



And from that the factor of safety expression was given by Newmark, considering only horizontal seismic acceleration like this. And later on people have also modified it considering vertical seismic acceleration as well. In this case, Newmark introduced another terminology which is known as yield acceleration, what is yield acceleration? It is that value of acceleration which makes the slope factor of safety equals to 1. So, at factor of safety equals to 1, you can find out this value of K h that will give you the value of K y. And if your actual seismic acceleration at a site, for which you are designing your slope, is less than that K y value; that means, automatically your factor of safety is more than one, you need not to worry about the displacement of the slope.

But if the seismic acceleration at the site is more than this critical seismic acceleration or yield seismic acceleration, then of course, your slope is going to fail, because factor of safety will be less than one in that case, and how much will be the displacement, that needs to be estimated. So, that estimation is done like this, relative acceleration which is causing the displacement, can be estimated as actual acceleration at the site, minus the yield acceleration. Now, if that value of acceleration if you integrate over the time, for which you are analyzing it, you will get the relative velocity. On further integration of this relative velocity over the time scale, you will get the relative displacement of your soil slope. This is the way; you get the displacement aspect also, in this Newmark's sliding block method.

(Refer Slide Time: 08:32)

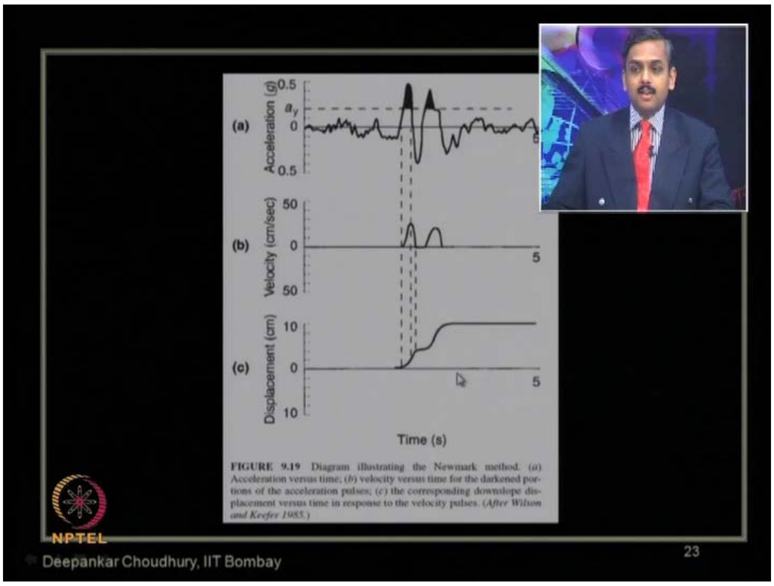


So, this picture shows the result for a soil block, which is failing or sliding like this, on a stable slope of 20 degree. And using the factor of safety value equals to 1, you can see this dotted line. The factor of safety curve for different values of phi value of soil is plotted over here using the, equation this one, for different values of K h. So, if you use different values of K h, and if you know this beta value equals to 20 degree. You can easily find out what will be the factor of safety; that you can easily plot for different values of phi. So, that is what has been done here, the plot of different values of factor of safety with different K h value, as a input and phi values as input, these curves have been plotted. Now, how to get the yield acceleration, if you now put this dotted line which is equals to factor of safety equals to one. If you, wherever it intersects the corresponding line; like for phi equals to 20 degree it is always failing, can you see that.

Whereas, for phi equals to 30 degree, it is stable up to the acceleration of this much value, which is say 0.165 or 0.17 something like that. In between 0.15 and 0.2, but beyond that, if your acceleration is there, in that case its factor of safety will be less than 1. In that situation it will be having displacement; that displacement you can compete using that Newmark’s approach. And this value, where the factor of safety equals to 1 it is nothing but your yield acceleration. Similarly, for phi equals to 40 degree it will be stable, up to a seismic acceleration value of close to about 0.35 or so. Beyond that value, it will be unstable, then it will start sliding. And how much is the displacement again you can calculate using this yield acceleration value, that is at which factor of safety equals to

1. So, this is the way how the calculation of factor of safety, and displacement calculation, using new Newmark's sliding block method is carried out.

(Refer Slide Time: 11:04)

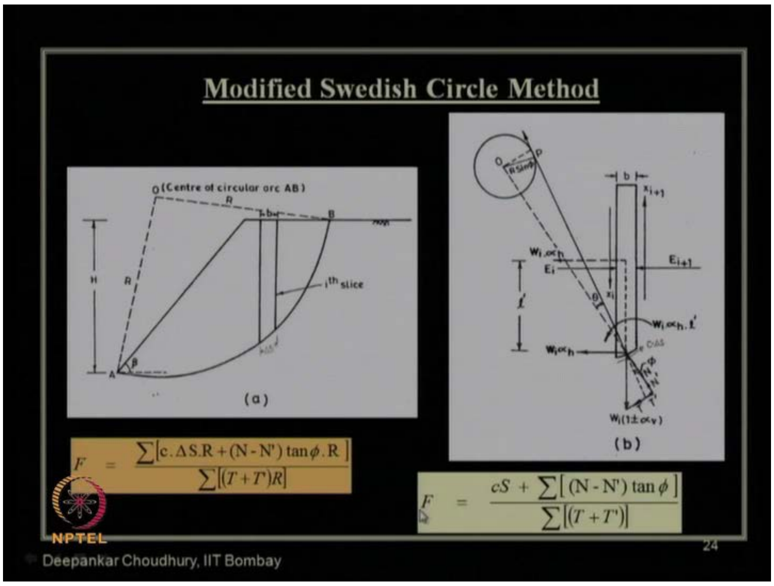


This is another example you can see, this is your yield acceleration. Suppose this is the profile of your acceleration verses time, this profile is known to you. You have now computed what is your yield acceleration value a y, for which the slope is having factor of safety equals to 1. Now on top of that, above that value of a y, whatever the portion, those shaded area, these black shaded areas are nothing but responsible for failure of your slope, or sliding of your slope. So, how much will be that sliding, because this region where the acceleration is there, it is much below than a y. So, obviously your slope is stable in this acceleration region. So, now, if we integrate it, we will get the velocity profile over the time scale. So, if you integrate between this time to this time. You need not to integrate from here to here.

Remember, what is the time scale we mentioned, for this integration; it is nothing but that time scale, where it exceeds that yield acceleration. And whenever it is again below the yield acceleration, you need not to consider that time scale, is it clear. That means, you are integrating over this time scale, and again you are integrating over this time scale. So, that is how, the velocity you are after integrating you are getting this one, and for this portion also you are getting velocity like this. Other places you do not get any velocity, because it is well below the yield acceleration, nothing to get integrated.

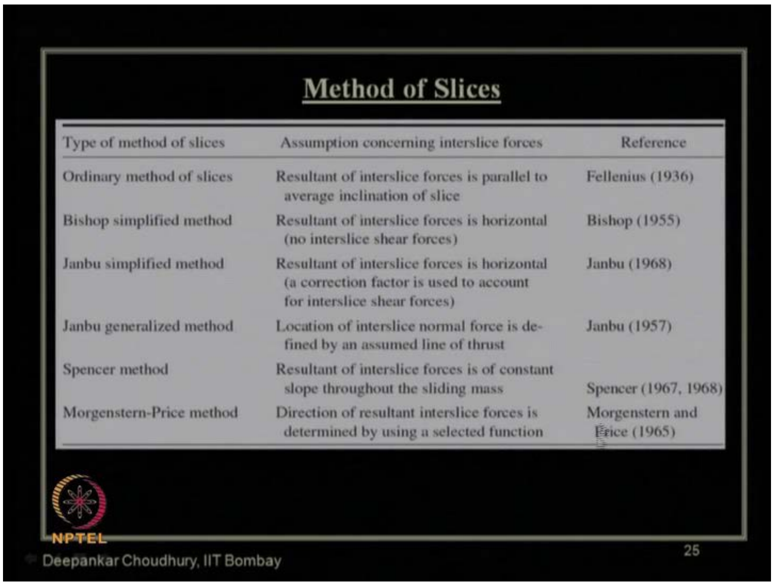
Similarly how you are getting the displacement, now you have to further integrate this zone, and get the displacement, over that time scale. But here you remember it is additive, because displacement is additive. So, once you have this displacement, the next time step when you are starting your calculation of displacement, the previous displacement has already occurred.

(Refer Slide Time: 13:18)



So, on top of that you have to add. So, that is why the displacement curve is like this. Now let us move to modify Swedish circle method, Swedish circle method all of us are aware about. This is the vertical slice; this is the typical factor of safety expression for the Swedish circle method. This is the circular arc of a failure surface, which we consider for slope stability analysis. Now what that are the additional thing in this case, in modified analysis, why it is modified, because in this case if you want to consider pseudo-static acceleration you can take this W i times alpha h for each slice. As well as the vertical component also this alpha v you can take; that is K h or K v in horizontal and vertical direction seismic acceleration, you can add to your equations and get the factor of safety modified value.

(Refer Slide Time: 14:05)

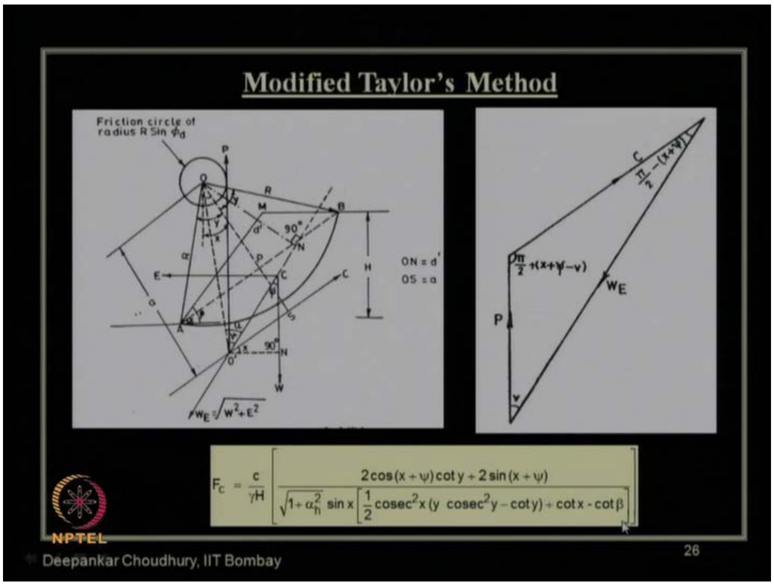


Method of Slices

Type of method of slices	Assumption concerning interslice forces	Reference
Ordinary method of slices	Resultant of interslice forces is parallel to average inclination of slice	Fellenius (1936)
Bishop simplified method	Resultant of interslice forces is horizontal (no interslice shear forces)	Bishop (1955)
Janbu simplified method	Resultant of interslice forces is horizontal (a correction factor is used to account for interslice shear forces)	Janbu (1968)
Janbu generalized method	Location of interslice normal force is defined by an assumed line of thrust	Janbu (1957)
Spencer method	Resultant of interslice forces is of constant slope throughout the sliding mass	Spencer (1967, 1968)
Morgenstern-Price method	Direction of resultant interslice forces is determined by using a selected function	Morgenstern and Price (1965)

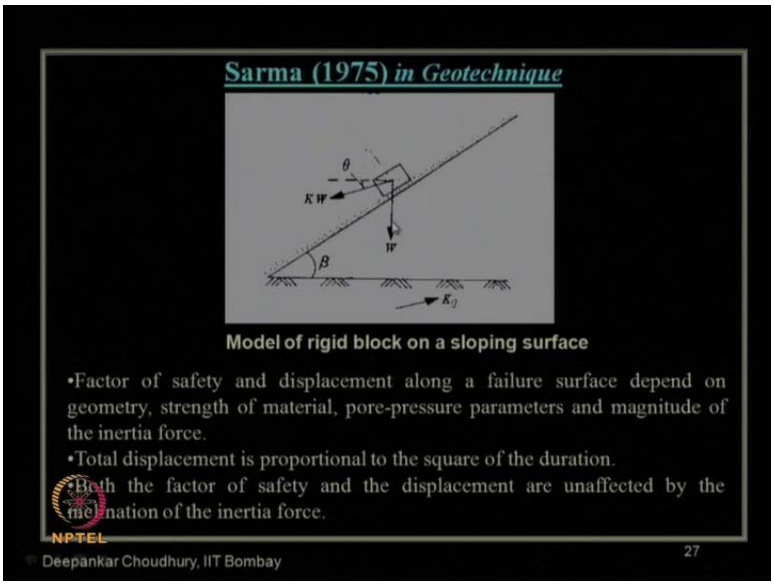
Like method of slices, we know these are the various methods of slices which are used in static case of slope stability analysis, and their references also are given over here.

(Refer Slide Time: 14:17)



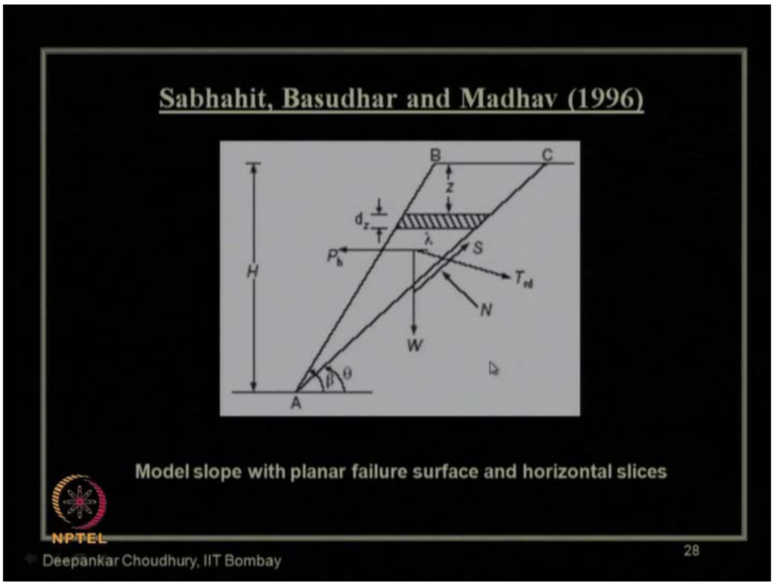
Now modified Taylor's approach, what does it says. Taylor's method of slop stability also your aware about, from you geotechnical engineering or soil mechanics course. So, this is the failure surface, which the normal to this or resultant to this failure surface should pass thorough, a circle which is concentric about this point O, so that is the Taylor circle isn't it. So, in this case, if you want to modify it, your W will change to now W E. W E is resultant W, which takes care of your horizontal seismic inertia force of E also, can you see that. So, your force polygon will change now into this shape, and by considering that your factor of safety, in terms of cohesion you can obtain like this.

(Refer Slide Time: 15:17)



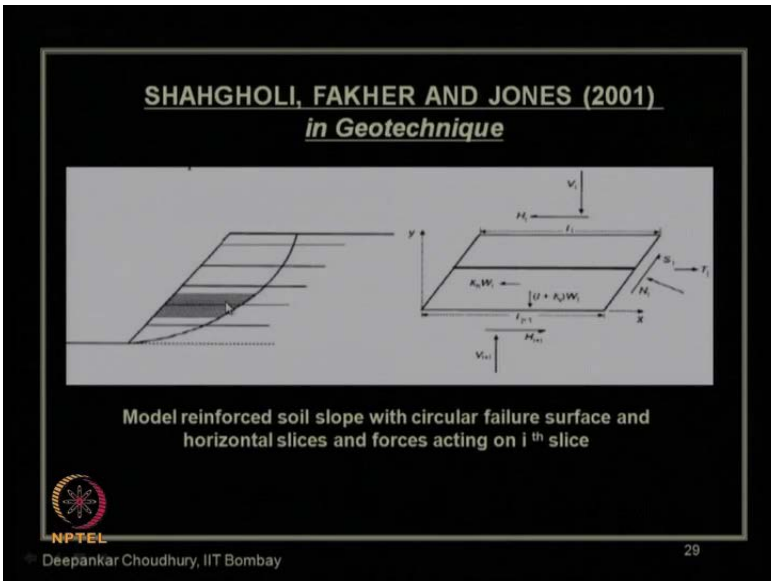
Now Sharma in 1975 proposed, or I should say extended the Newmark's sliding block model, for a rigid block on a sloping surface like this. And he gave the solution, using pseudo-static approach. This solution is available in the journal paper geo-technique, published by IC London. Factor of safety and displacement along a failure surface depend on the geometry strength of the material, pore pressure parameters, and magnetite of the inertia force. And total displacement is proportional to the square of the duration that we know, because you are integrating it two times. Both the factor of safety and displacement are unaffected by the inclination of the inertia force; that is what Sharma in 1975 has proposed.

(Refer Slide Time: 16:08)



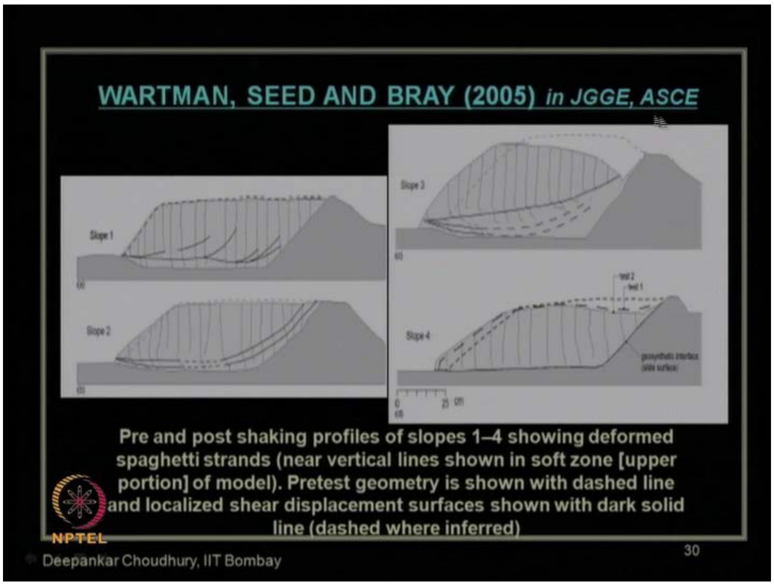
Other researchers like, Sabhahit, Basudhar and Madhav in 1996 proposed this horizontal slice method, for slope stability analysis. And this is the additional horizontal force of p h, which is acting along with other static forces.

(Refer Slide Time: 16:31)



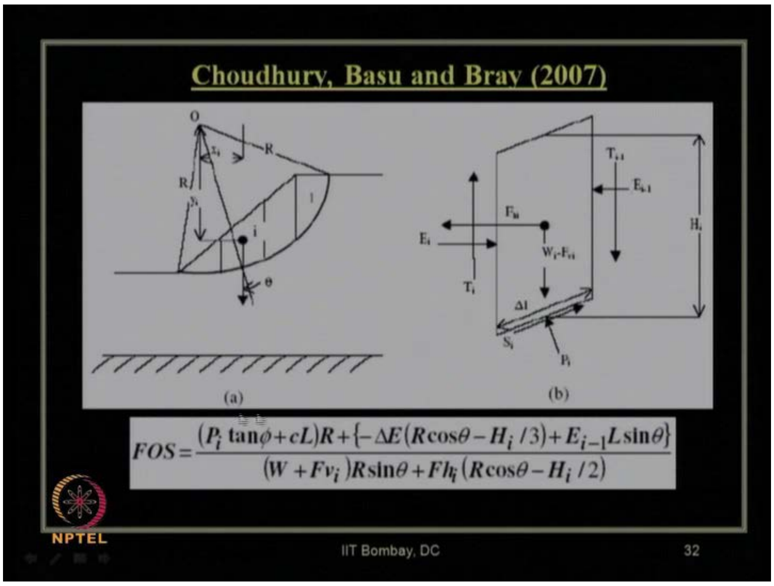
Then Shahgholi Fakher and Jones in 2001, they also use the horizontal slice method for slope stability analysis for a reinforced soil slope. So, for reinforced soil slope, this paper is available again in the journal Geo-technique, published by IC London. This is the basic force diagram on an infinite simal soil mass, which contains only one reinforcement like this. So, for reinforce soil slope, this is the analysis proposed by Shahgholi et al, using horizontal slice method.

(Refer Slide Time: 17:09)



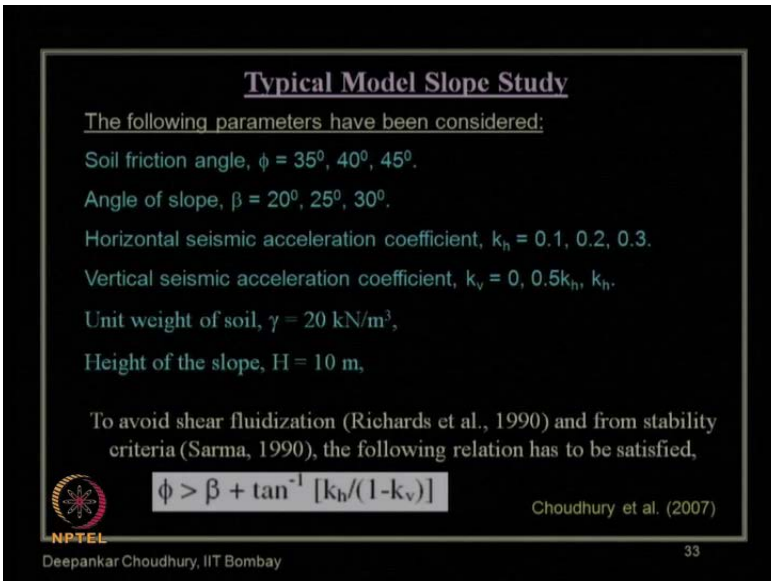
Other researchers like Wartman, Seed and Bray in 2005. They had shown from shake table test and numerical analysis; that is pre and post shaking profile of different slopes, which are shown in this figure over here, and their corresponding displacement. So, you can see the displacement profile, how this things have moved from original position to. So, these are the different steps you can see. This paper is again available, in the journal of Geo-technical and Geo-environmental engineering of ASCE.

(Refer Slide Time: 17:48)



Next work was done by Choudhury Basu and Bray in 2007 Choudhury et al. This is the publication detail. This is the work done by one of my master student Somdatta Basu, along with my collaborator from, university of California at Barkly in USA, Prof. Jonathan D Bray. So, three of our work has been published in this geotechnical special publication of ASCE. This is the paper name 2007, page numbers etcetera. So, we have analyzed, a soil slope considering a arc of an failure surface, considering this vertical slice method. And the horizontal additional seismic inertia force, as well as vertical seismic inertia force are also taken care of, including the interface this between slices what are the interface, forces which are acting. So, using that, the factor of safety expression, the closed form solution we have expressed for any ith slice, any slice in this fashion, in this form.

(Refer Slide Time: 19:06)



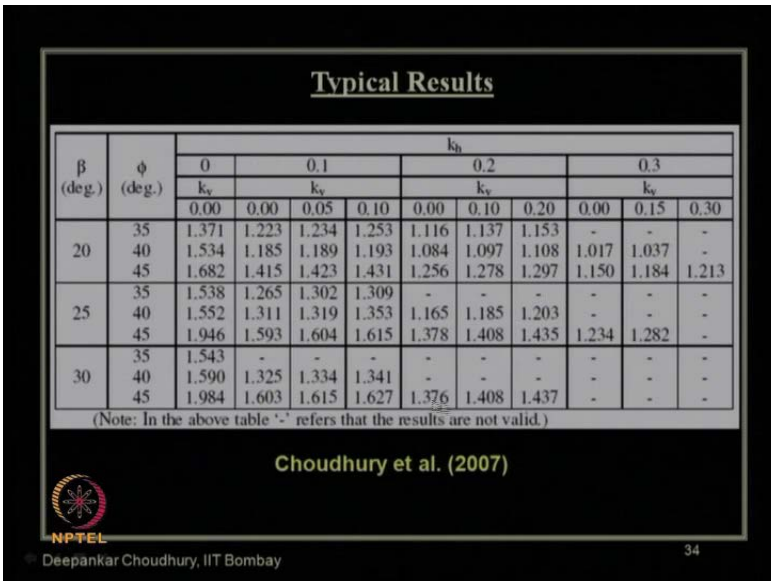
And then, a parametric study has been considered for various soil friction angle 35, 40, 45, with various angle of slope. And remember in this case, as proposed by Richards et al in 1990 to avoid the phenomenon of shear fluidization, what is shear fluidization I will explain now. And also from the stability point of view, as proposed by Sharma in 1990, what does it say for stability. The soil friction angle phi valve must be greater than, beta is the slope angle of the ground, plus tan inverse of k h by 1 minus k v. This k h is horizontal seismic acceleration, k v is vertical seismic acceleration. So, this part tan inverse k h by 1 minus k v is coming from Richards et al 1990. This paper is available in the journal of geotechnical engineering ASCE, to mention the concept of shear fluidization.

So, what Richards et al said. They motioned that, even a dry cohesion less soil, like we know about the flowing of a soil, after the liquefaction occurred in case of cohesion less soil, saturated condition. But if the cohesion less soil is completely dry, then also it can flow like a fluid. In what way it can flow like a fluid. Let us say, let us take a plane like this, if we place some dry sand on this plane, then it will stack up to a certain height; that will be its angle of repose that we know; that is under static condition, its stable. Now let us start shaking this, if you shake it like this, what will happen, slowly it will start spreading out; that means it starts flowing. Why it is happening, because there is a shaking which causes the soil grains to fail.

So, that value of internal friction angle between the soil grains which is phi. The tan of phi value should be greater than, that component of k h by 1 minus k v, where k h is your horizontal seismic acceleration coefficient, and k v is your vertical seismic acceleration coefficient. that phenomenon which occurs in dry cohesion less soil; that is called shear fluidization, if the soil starts behaving like a fluid or flows like a fluid. That means, for instability phi will be less than this component, and for stability phi must be greater than this one. So, in all your pseudo-static design, what I will say, you must always check this criteria; that is whether, your soil which you are using, for your any analysis so far I have explained, slope stability analysis, retaining wall analysis, shallow footing design.

In all this spaces, you need to consider, even when we are considering the dry soil case; that whether the soil material itself is stable under that seismic condition or not, because at very high value of shaking, the soil itself will start flowing like a fluid; though it is there is no presence of water, so that shear fluidization criteria has to be satisfied. And where from this beta comes from, as Sharma has combined this two effect, this beta comes from the static criteria of slope stability. As you know for cohesion less soil, to have a stable or finite soil slope which will be stable, what is the criteria. The phi value of soil must be greater than the soil slope beta, isn't it; otherwise it will not be stable. That is the portion from the criteria of stability is concerned, that needs to be added with respect to this seismic component also. So, for stability criteria of soil slope, phi value has to be greater than this phi beta plus tan inverse k h by 1 minus k v.

(Refer Slide Time: 23:45)



β (deg.)	ϕ (deg.)	k_h									
		0	0.1			0.2			0.3		
		k_v	k_v			k_v			k_v		
		0.00	0.00	0.05	0.10	0.00	0.10	0.20	0.00	0.15	0.30
20	35	1.371	1.223	1.234	1.253	1.116	1.137	1.153	-	-	-
	40	1.534	1.185	1.189	1.193	1.084	1.097	1.108	1.017	1.037	-
	45	1.682	1.415	1.423	1.431	1.256	1.278	1.297	1.150	1.184	1.213
25	35	1.538	1.265	1.302	1.309	-	-	-	-	-	-
	40	1.552	1.311	1.319	1.353	1.165	1.185	1.203	-	-	-
	45	1.946	1.593	1.604	1.615	1.378	1.408	1.435	1.234	1.282	-
30	35	1.543	-	-	-	-	-	-	-	-	-
	40	1.590	1.325	1.334	1.341	-	-	-	-	-	-
	45	1.984	1.603	1.615	1.627	1.376	1.408	1.437	-	-	-

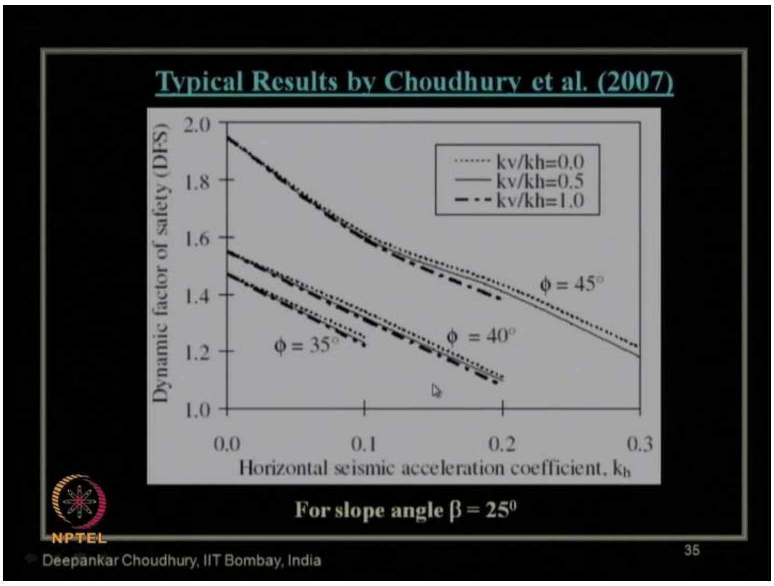
(Note: In the above table '-' refers that the results are not valid.)

Choudhury et al. (2007)

NPTEL
Deepankar Choudhury, IIT Bombay
34

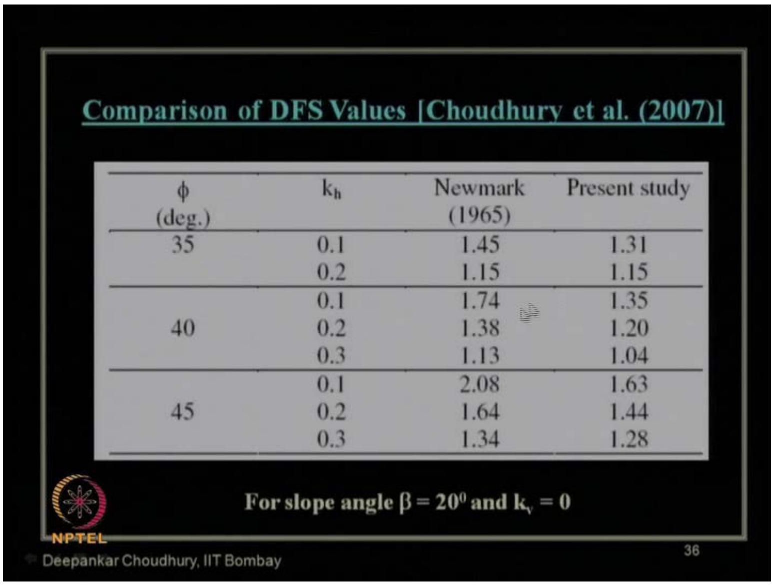
So, with these parametric variations, we obtain the typical results for different soil slope angle, with different values of friction angle of soil as I have mentioned, with k h value 0 k v value 0 means. These are factor safety under static condition, and these are the values of the factor of safety under seismic condition, with different values of k h and k v. You can easily see the critical value of factor of safety is keep on decreasing, as the seismicity increases in this value of k h also increasing, or when the value of k v also is increasing; that can been seen very clearly. In both the cases, it is going to give us the more critical state, or more towards the unsafe side, I will say, when the seismicity value increases, in terms of horizontal and vertical seismic acceleration. And this dash value shows, either this is not a stable condition in terms of shear fluidization criteria or stability criteria.

(Refer Slide Time: 24:56)



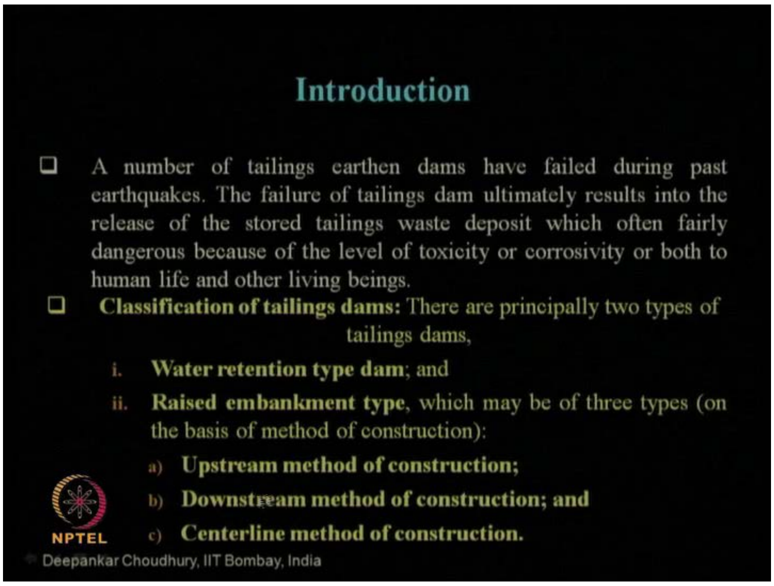
This is the plot of the dynamic factor of safety, in terms of k h and k v for different values of phi.

(Refer Slide Time: 25:03)



This is the comparison of our present study; that is the study by Choudhury et al 2007 with the Newmark's sliding block method. Of course, we considered only the stable slope. So, you can see, in present study we can still get, a further lower value of factor of safety or critical value of factor of safety, because of consideration of all both the things.

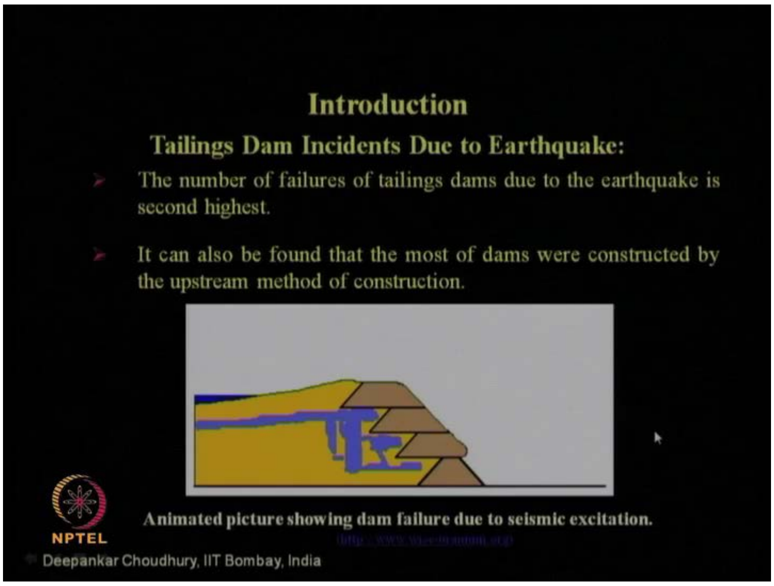
(Refer Slide Time: 25:41)



Now, let us come to another sub topic, which is seismic stability of tailing dam. Now what is tailing dam or earthen dam, let us first introduce. A number of tailings earthen dam have failed during the past earthquake. The failure of tailing dam, ultimately results into the release of, stored tailings materials or waste deposit, which are often fairly dangerous, because of its level of toxicity or corrosivity or both, to the human life or other living beings. As we all know, first of all earth dam design is an important structure design, because if failure of dam occurs, then there will be a huge calamity, because on the downstream side, whoever lives there, entire thing will get washed out. So, that is why design of earth dam is a very important design, for which we should take all precautions. More so for the tailing dam, what is tailing dam. Tailing dam is nothing but the dam, which stores the tailing materials, or it can be a waste material, it can be nuclear waste, it can be other dumping waste, which are getting stored over there. So, you can imagine, if the tailing dam fail, there will be much more disaster than earthen dam, because not only the downstream site gets washed away, but also those storage materials are getting spread over the entire locality in the downstream.

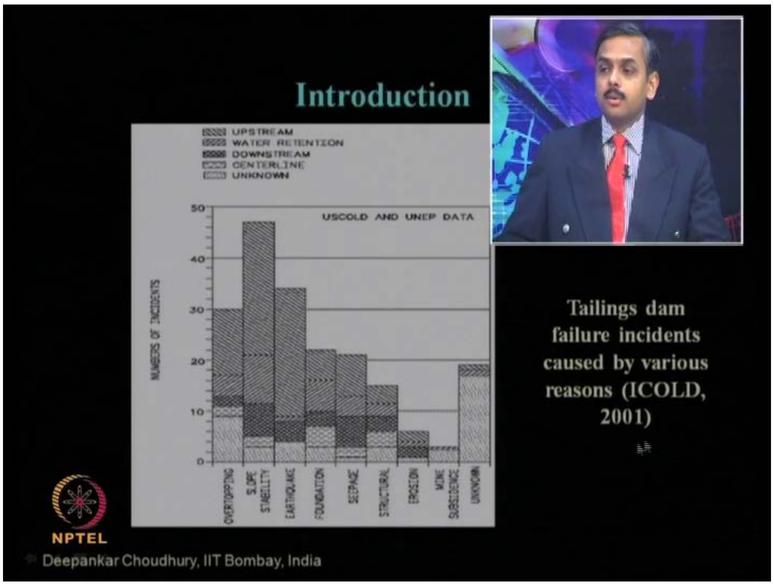
So, that is why design of this tailing dam is very important in these seismic zones, or seismic conditions; and it need to be carried out carefully. Now classification of tailing dam; there are majorly two types of tailing dam, one is called water retention type dam; that we commonly know about this dam, water retention type dam. Whereas, another one is raised embankment type, raised embankment type tailing dam. It has been constructed in three different ways; that is there are three approaches based on that, their sub classification as been made; one is called upstream method of construction, another is called downstream method of construction, and the other one is called central line method of construction. Among this three, most commonly used two things are upstream and downstream method of construction.

(Refer Slide Time: 28:10)



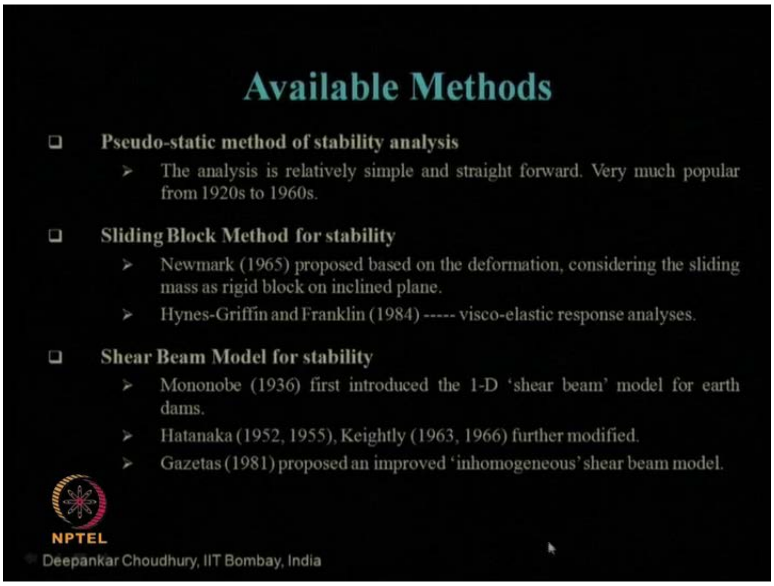
Let us see over here, this is the failure of tailing dam, which is shown through this animated picture. The number of tailing dams failed in the earthquake. The total number is second highest in the world, as far as statistics is concerned, data available in the literature is concerned, I will show that very soon. It can also be found that, most of the dams were constructed by the upstream method of construction, as I said upstream method, and another common method is downstream method. This shows during the seismicity, once it fails, the entire downstream get washed away. Initially your downstream was existing, but now if everything goes out, then nothing left in the entire region.

(Refer Slide Time: 28:57)



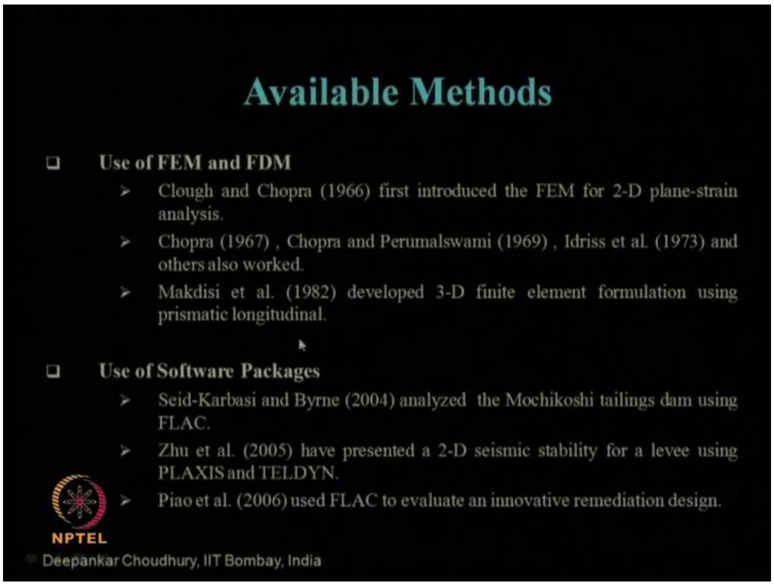
This is the statistics, look at here; the tailing dam failure incidents, which are caused by various reason. There are other reasons also, as you know by which tailing dam can fail; earthquake is one of them, and that is the second highest, as I said just now, this is as far ICOLD 2001 data. You can see the various numbers of incidents, and what are the different reasons for which it is failing. Like over toping is one reason for the dam failure, slope stability is another reason, many a times slope is not a stable one, earthquake is another reason, foundation problem is another one, seepage problem is another one, structural failure of the dam, then erosion, then mine subsidence, and there will be always some unknown reasons as well. So, among these, you can see the highest reason is the slope stability failure for dam, and the second highest is the earthquake, as per this publication.

(Refer Slide Time: 30:03)



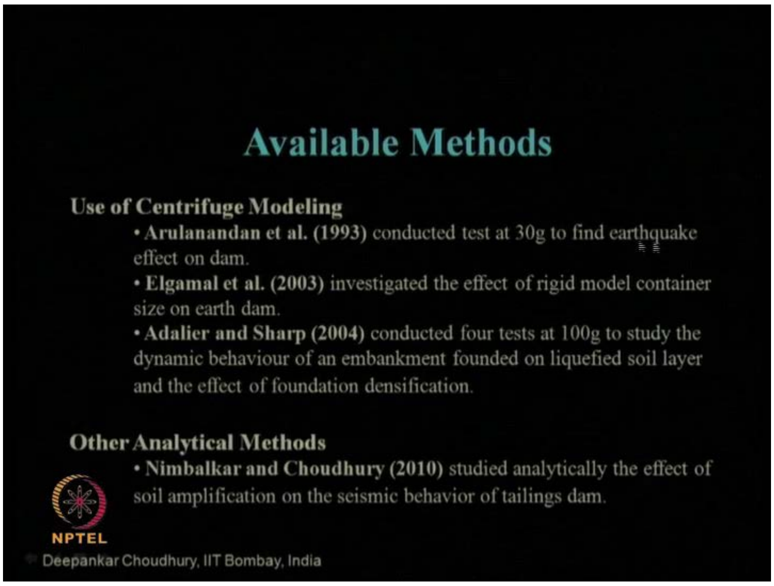
Now, what are the available methods let us look at it first, for the tailing dam design. Basic one is pseudo-static method of stability analysis, it is very straight forward and simple, we have already discussed. For that what people use, they use Newmark's sliding block method, because it gives not only your factor of safety, but also if it displaces, how much will be your displacement, that can be estimated. And shear beam model for stability is another method, which has been proposed by Mononobe in 1936, by using one dimensional shear beam model, and later on other researchers have extended it, like Gazetas in 1981, for in homogenous shear beam model he had proposed.

(Refer Slide Time: 30:48)



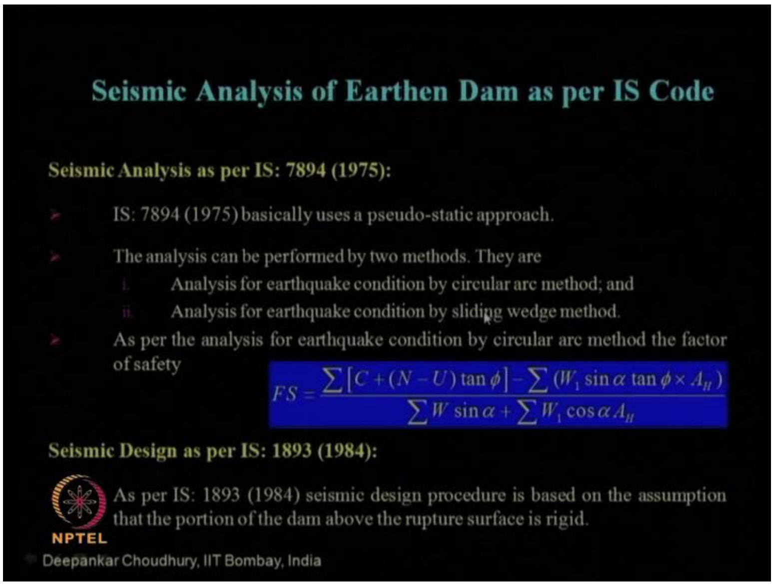
Other methods are; like finite element method, finite difference method, like Claugh and Chopra in 1966, first introduced the finite element method, for two dimensional plan strain analysis for dam. And later on, other researchers, the latest one is Makdisi et al in 1982, developed 3 dimensional finite element formulations for this earthen dam. And uses of various software packages are now-a-days available, as you know FLAC, which is a finite difference based software. The full name is FLAC is, Fast Lagrangian Analysis in Continues, which is a very robust geotechnical software. Other geotechnical software's are like, Plaxis, Teldyn, Taldrain, Geo slope. There are many other slope related software, which can do the slop stability analysis. And Piao et al in 2006 use the FLAC to evaluate the innovative remediation design for this earthen dam.

(Refer Slide Time: 31:59)



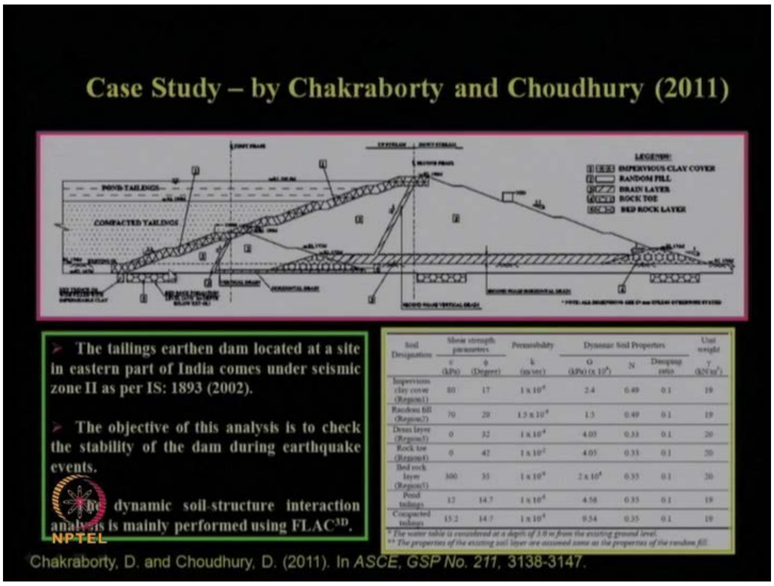
Other available methods in terms of experiments are concerned, like centrifuge modeling is one of the options, as we all know. For geotechnical structures, in this earthquake condition you can model a dam, and then give input seismic accelerations, to find out the response of it. So, Arulanandan et al in 1993 conducted a series of test, at 30 g level, to find out the effect of earthquake on dam. Later on, Elgamal; Prof. Ahamad Elgamal, who is now a Prof. at University of California, at San Diego, earlier he was Prof. at R P I, New York; this work refers to his R P I work, because they had a national centrifuge in USA, which can carry out the dynamic test or earthquake test. So Elgamal et al 2003, they investigated the effect of rigid module container size on the earthen dam. They have also conducted the earthquake test on the model earthen dam. Then other researchers, like Adalier and Sharp in 2004 conducted four tests at 100 g level, in centrifuge, to study the dynamic behavior of an embankment founded on liquefied soil layer, and the effect of foundation densification. There are other analytical methods available, as proposed by Nimbalkar and Choudhury, by my first PhD student Dr. Nimbalkar. We also worked analytically, to mention how the soil amplification factor involves or affects the behavior or seismic behavior of the tailing dam, or the earthen dam.

(Refer Slide Time: 33:47)



Let us now look at; what are the recommendations, provided by our Indian design code for design of this earthen dam, under seismic condition. So, seismic analysis as per IS code 7894 of 1975 version. It basically proposes that pseudo-static approach needs to be used. The analysis can be performed by two methods, and those two are; either you can use circular arc method or the sliding wedge method. And as per the analysis of earthquake condition, the circular method, the factor of safety is given by this expression. So, if somebody is using the circular arc method, they can use this factor of safety. If somebody is using sliding wedge method, they can use the Newmarks sliding block method basically. And what is the seismic design criteria as per IS 1893, as I said 1984 is the latest version for the geotechnical structure design till date. Seismic design procedure is based on the assumption, that the portion of the dam above the rupture surfaces is rigid.

(Refer Slide Time: 35:06)

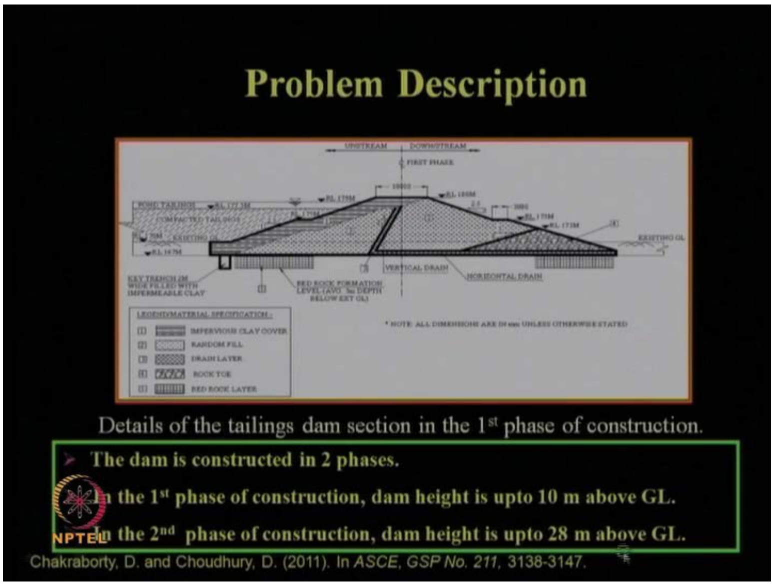


Now, let me explain to you one case study, which I had conducted at IIT Bombay. This is through a sponsor research project, form atomic energy regulatory board, government of India. So, through that project, we also studied some sample problem, not exact problem. And also we have calculated the factor of safety, and the seismic behavior of actual field problem. So, both we have studied; a sample problem, or customized problem, as well as the actual real problem. So, this work was done by Mr. Debarga Chakraborty, who did Masters with me, under my supervision, at IIT Bombay. So, his master thesis was completely on this project.

So, Chakraborty and Choudhury 2011 publication gives this all details. The detail can be obtained in this paper; this is ASCE geotechnical, special publication number 2011. This is the page number. One can easily find out in ASCE library, this ASCE paper. So, what was the problem definition, this is the actual site, that a tailing dam has to be constructed in the eastern part of India, which comes under zone number two. So, site was selected, based on the selection of the site it was already mentioned it will be in zone two, which is the lowest hazardous zone, as per as our IS 1893 part 1 2002 is concerned, that we have already seen.

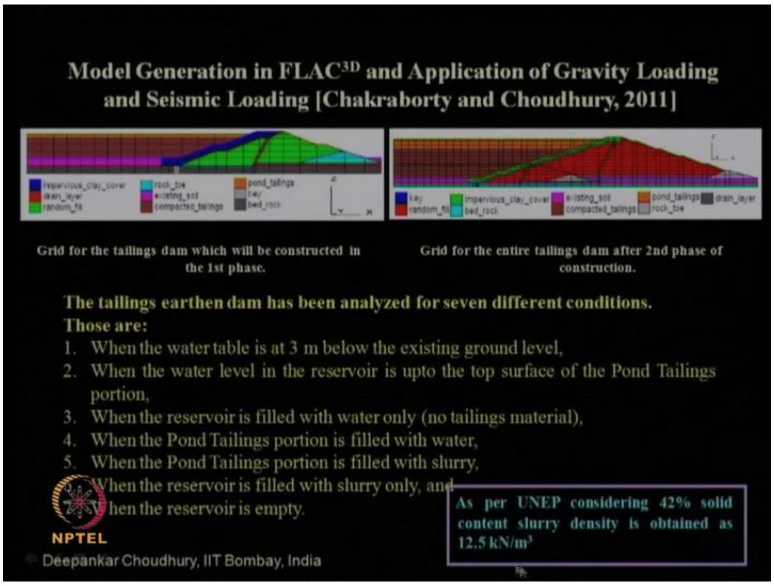
So, in that zone, it is our objective was, to check the stability of that tailing dam, under earthquake events, and dynamic soil structure interaction analysis was performed using this, finite difference based software FLAC 3D, 3D means 3 dimension. In 3 dimension we use this, because all the dimensions of the dam were given by the concerned agency, and these are various input values of soil parameters, dynamic soil parameters, static engineering properties of soil etcetera, for different zones, of this tailing dam. Remember can you see here, this two phases, this first phase is the first phase of the dam, which is of about 10 meter height. Later on it has to be proposed to extend it to second phase which is 28 meter of the height of the dam, and it will be constructed in downstream method of construction, as you can see here, it is a downstream method of construction. And on the upstream side of this tailing dam, what are the things to be stored. It is proposed that compacted tailing material will be stored, which are non hazardous waste material, nuclear waste material. Non hazardous nuclear waste has to be stored here, and this is the top pond tailing portion.

(Refer Slide Time: 38:19)



So, using this data, as I said dam is proposed to be constructed in two phases. So, we need to consider, the stability aspects and seismic behavior of the dam, both for first phase as well as for second phase, because second phase will be constructed, whenever there will be a requirement of storing those tailing waste material over the time. So, initially first phase will be constructed, after few years second phase will be constructed. So, 10 meter above ground level is in the first phase, and 28 meter above ground level is in the second phase.

(Refer Slide Time: 38:57)

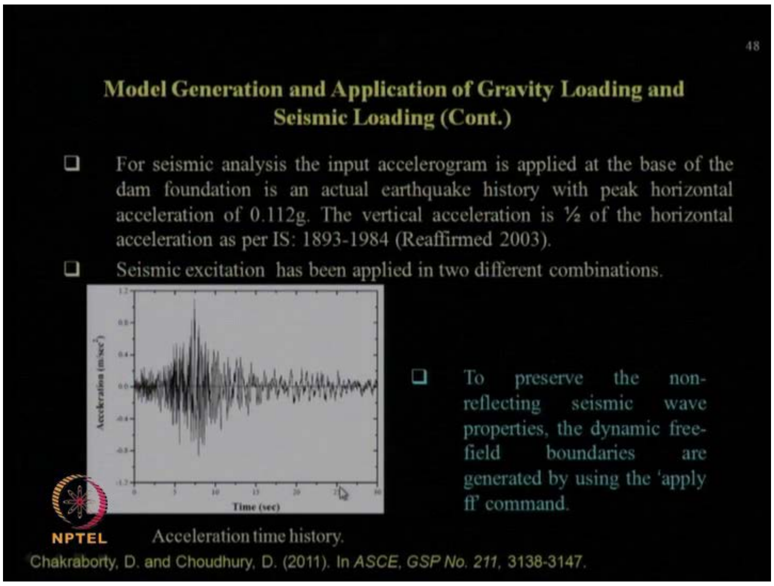


So, this is the FLAC three dimensional modeling, in. This is the machine; you can see the grid line. First it has to be loaded with the gravity loading, because static stability first thing has to be observed, with the condition of the tailing material. Remember from simple earthen dam it is simple, because it is no longer only water. So, you have not only typical four cases, as we have for the case of earthen dam design. Here, we have seven different cases, which are possible to arise, or seven different conditions, what are those seven different conditions. Like when water table is 3 meter below the exiting ground level, that is, this is your existing ground level below that 3 meter is your water table, this is fully dry, when the water table is the reservoir up to the top surface of the tailing pond portion; so this tailing pond portion up to top is your reservoir water. Another is when the reservoir is filled with water only; that is no other, nuclear waste material or waste material is dumped here.

Because then problem will change, because water is having one unit weight, another dumped material, will having another unit weight. Now fourth condition is, when the pond tailing portion is filled with water only; that is it has circled, and pond tailing the top portion is filled with fresh water. Then fifth condition is when pond tailing portion is filled with slurry; that means, when your waste material has not settled, it is still in slurry form, the pond portion is not yet fresh water. Sixth point is, when the reservoir is filled with slurry only, entire reservoir filled with slurry. And seventh condition is, when the entire reservoir is empty. So, all conditions we have analyzed, and these are the as per as

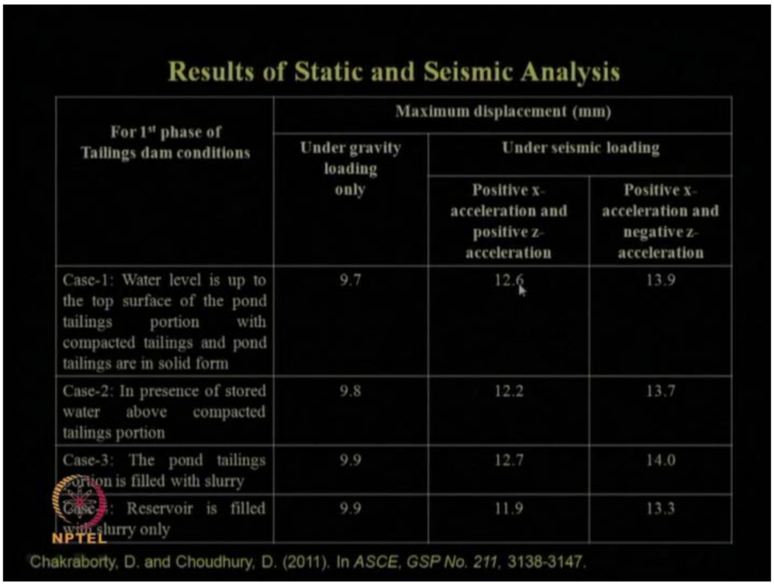
UNEP guideline for 42 percent of solid content in the slurry, as far as the material which has to be deposited form that we found out, this is the density to be considered.

(Refer Slide Time: 41:07)



Then, what value of earthquake need to be chosen, we have chosen Taft earthquake acceleration time history, with peak horizontal value of this one, which reaffirmed with the IS code recommendations, and half of that value is used for vertical seismic acceleration for the pseudo-static analysis etcetera. In dynamic analysis you need not to assume any of this k h and k v value, as you know, because you are giving a full acceleration time history, as an input in your model, in your FLAC model, but that is your exact dynamic analysis. But when you are analyzing it using pseudo-static or Pseudo-dynamic approach, you need to take care of that k h and k v value. So, this is the input seismic acceleration as I have shown over here.

(Refer Slide Time: 41:59)



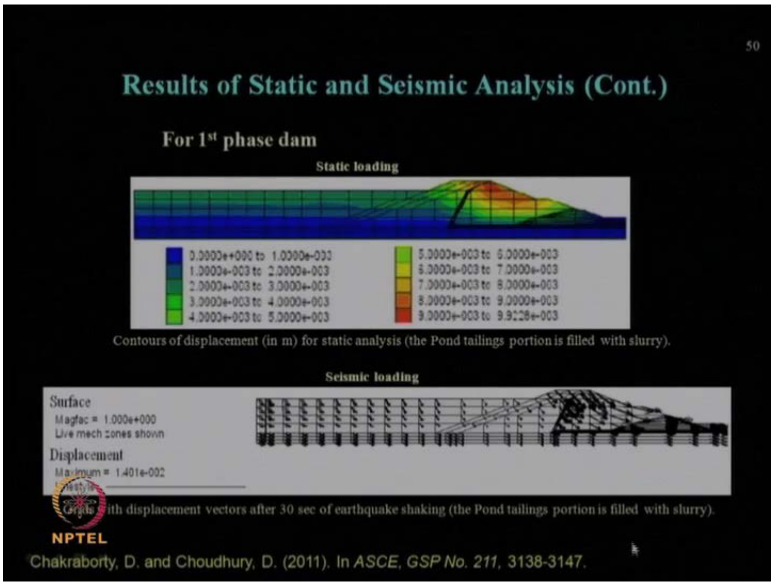
Results of Static and Seismic Analysis

For 1st phase of Tailings dam conditions	Maximum displacement (mm)		
	Under gravity loading only	Under seismic loading	
		Positive x-acceleration and positive z-acceleration	Positive x-acceleration and negative z-acceleration
Case-1: Water level is up to the top surface of the pond tailings portion with compacted tailings and pond tailings are in solid form	9.7	12.6	13.9
Case-2: In presence of stored water above compacted tailings portion	9.8	12.2	13.7
Case-3: The pond tailings portion is filled with slurry	9.9	12.7	14.0
Case-4: Reservoir is filled with slurry only	9.9	11.9	13.3

Chakraborty, D. and Choudhury, D. (2011). In *ASCE, GSP No. 211*, 3138-3147.

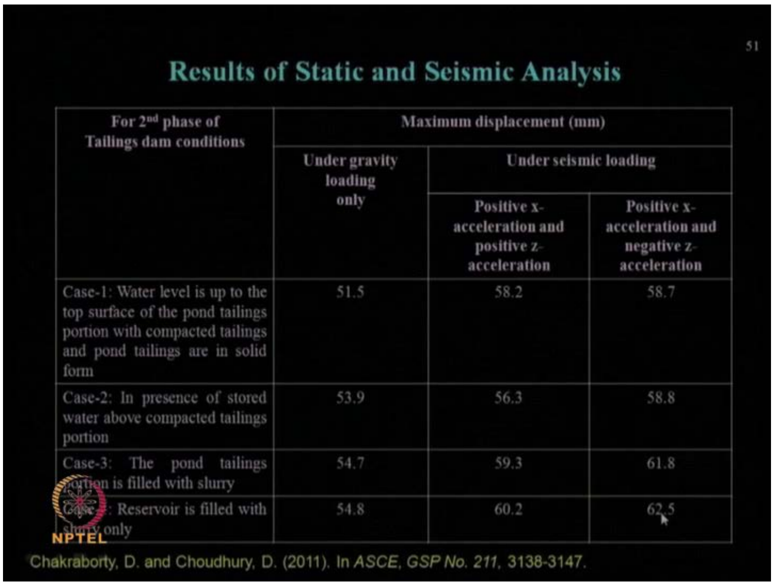
And these are the results; you can see this is for the first phase of tailing dam. In first phase various cases we have analyzed. Only four are shown over here, other three are available in the paper. So, one can refer this paper very easily. What is the maximum displacement in millimeter at the crest level, at the top level of that tailing dam, under gravity loading means under static condition, these are the values. And when the seismic loadings are acting, based on their different direction of working in horizontal and vertical direction combination, these are the values.

(Refer Slide Time: 42:38)



So, this is the mesh of displacement profile, you can see cantors of displacement profile of fast phase dam in FLAC 3 D under static loading. And this is the under seismic loading, how the vector representation displacement vectors, in which direction it is tending to move, that you can easily see. Obviously, it will try to fail over this portion, you can see from there also, slope stability failure. It depends of course, what type of condition you are considering. This is the condition in which this figure is shown. The pond failing portion is filled with slurry, for that combination. Like for all other 6 different combinations you will get different picture.

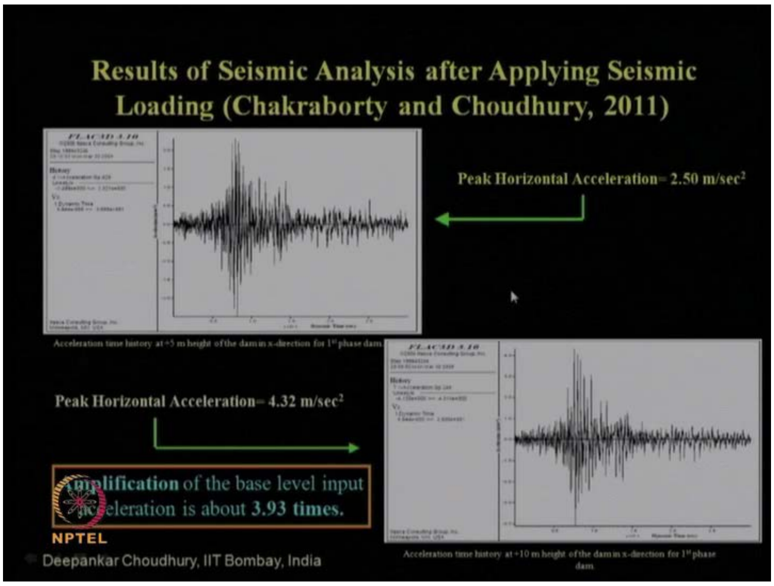
(Refer Slide Time: 43:25)



For 2nd phase of Tailings dam conditions	Maximum displacement (mm)		
	Under gravity loading only	Under seismic loading	
		Positive x-acceleration and positive z-acceleration	Positive x-acceleration and negative z-acceleration
Case-1: Water level is up to the top surface of the pond tailings portion with compacted tailings and pond tailings are in solid form	51.5	58.2	58.7
Case-2: In presence of stored water above compacted tailings portion	53.9	56.3	58.8
Case-3: The pond tailings portion is filled with slurry	54.7	59.3	61.8
Case[illegible]: Reservoir is filled with slurry only	54.8	60.2	62.5

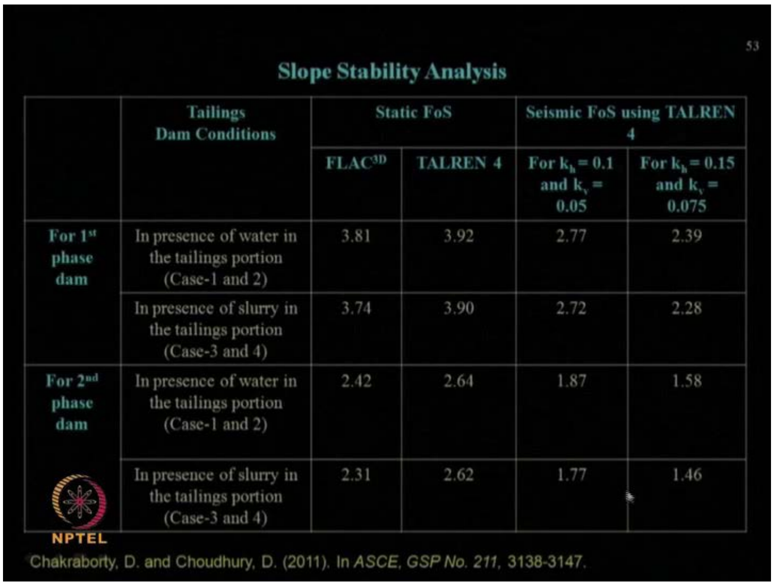
For the second phase of the tailing dam, the maximum displacement increased up to, in the range of 50s, 50 millimeters in terms of that, about 55 millimeter you can see over here, under static loading, and that further increased under seismic loading up to the value of maximum of 62.5 millimeter, under seismic condition.

(Refer Slide Time: 43:53)



Then, also this seismic output of acceleration verses time response at different height if this tailing dam is considered, why it is considered. To see how much is the amplification is occurring in that material of tailing dam. You can see, here peak horizontal acceleration is 2.5, as you can clearly find out from this output of this FLAC result. This is the acceleration verses dynamic time response, at a height of 5 meter. Remember our tailing dam height in this case first phase is, 10 meters. This is at 5 meter means, in between at the center point of the tailing dam. And this one shows the acceleration time history at the height of 10 meters; that means, at the top surface, or at the crest level of the tailing dam. So, what does it mean, it shows clearly that from the base input acceleration, which already we have seen over here, you will find out the p j value over here. So, this is your input base motion. Compared to that output at 10 meter level and at 5 meter level, you got these values, which are given over here. So, it shows clearly and amplification of about 4 times in that material. So, when you are considering your design of slope stability etcetera, you need to consider this one, but pseudo-static analysis cannot do that, only pseudo-dynamic can take care of this amplification.

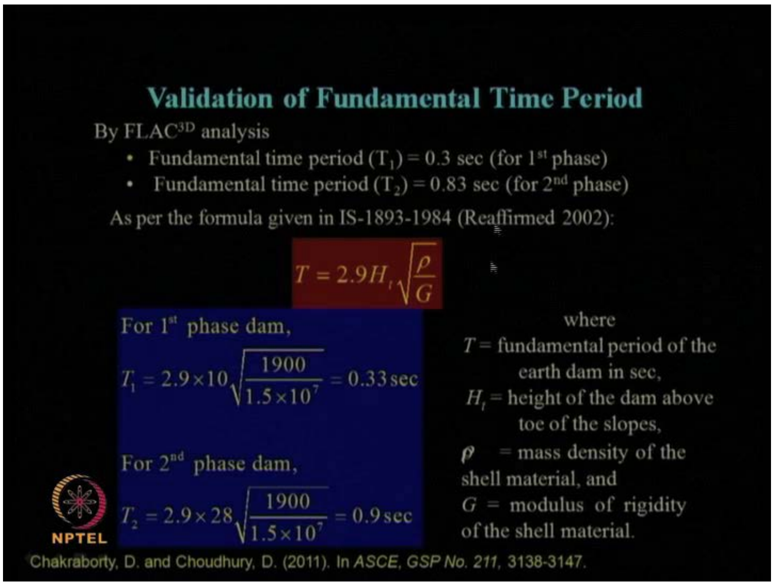
(Refer Slide Time: 45:38)



	Tailings Dam Conditions	Static FoS		Seismic FoS using TALREN 4	
		FLAC^{3D}	TALREN 4	For k_h = 0.1 and k_v = 0.05	For k_h = 0.15 and k_v = 0.075
For 1st phase dam	In presence of water in the tailings portion (Case-1 and 2)	3.81	3.92	2.77	2.39
	In presence of slurry in the tailings portion (Case-3 and 4)	3.74	3.90	2.72	2.28
For 2nd phase dam	In presence of water in the tailings portion (Case-1 and 2)	2.42	2.64	1.87	1.58
	In presence of slurry in the tailings portion (Case-3 and 4)	2.31	2.62	1.77	1.46

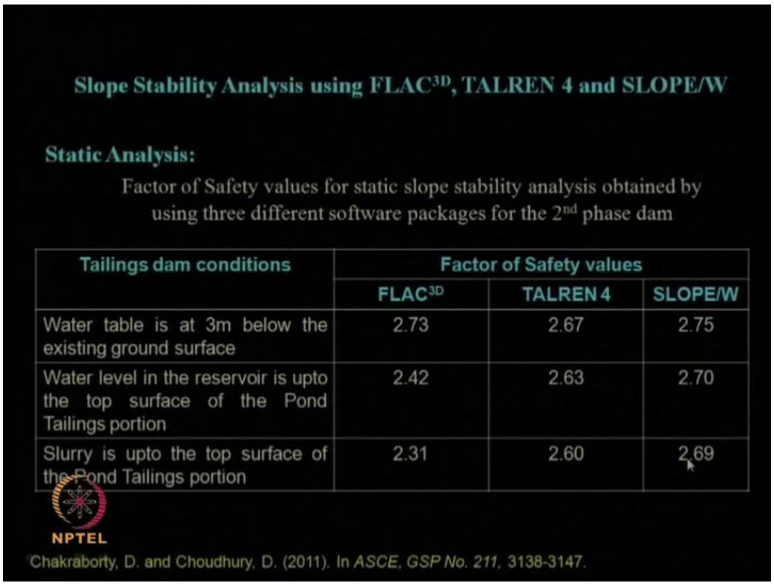
So, this is the slope stability analysis, using the software; that is FLAC 3D and TALREN for, using that for first phase and second phase of dam, for different cases or different combinations, the static factor of safety, as well as seismic factor of safety are obtained. So, if seismic factor of safety are more than 1.15, then it is safe. In all case we got it safe for the given input value.

(Refer Slide Time: 46:06)



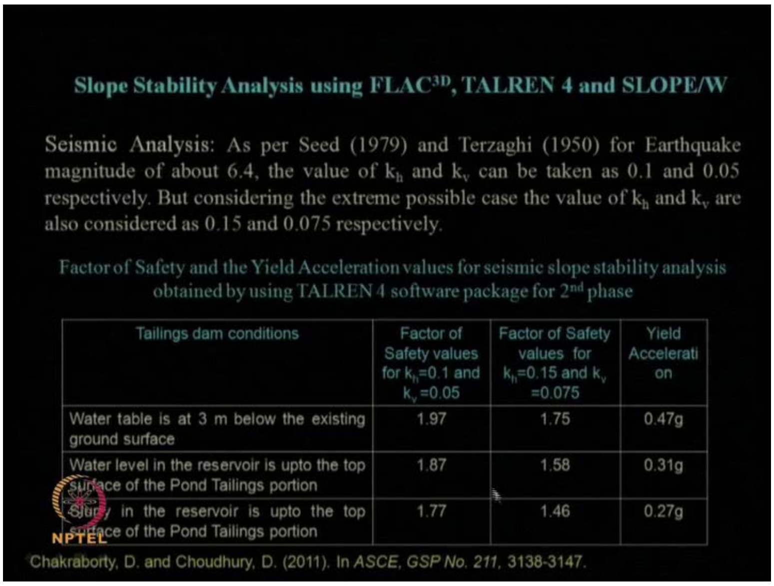
Next, we need to calculate, or validate the fundamental time period of the entire structure, how we do that. In the FLAC 3D analysis, we automatically get the fundamental time period from the analysis, for both first phase dam and second phase dam. These are the values of fundamental time period, which you can calculate as per IS code proposal; that is IS 1893, 1984. This is the equation how to calculate the fundamental time period of an earthen dam. So, for first phase of dam, putting this values 10 meter is the height, H T is 10 meter, rho is the density, mass density of the shell material, and G is the modulus of shear modulus of the shell material, using that you will get the value of fundamental time period, for first phase of dam, for second phase also similarly, using height as 28 meter. You can see, you can compare these values of 0.33 seconds, as obtained using this IS code value, and as obtained in the FLAC 3D analysis. They are quite comparable, also for the second phase of the dam.

(Refer Slide Time: 47:17)



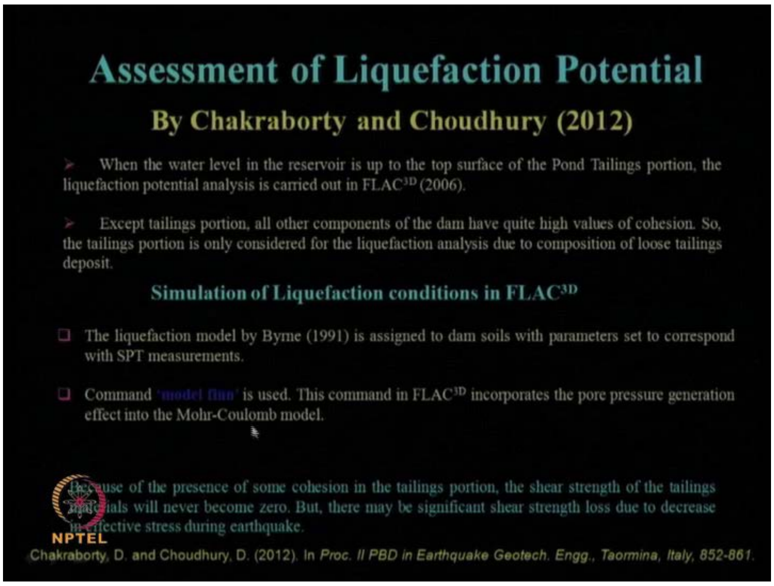
Now other results of static analysis in terms of factor of safety values, as I said, tailing dam condition FLAC 3D, TALREN and SLOPE W, these are common software used for the slope stability analysis as we know.

(Refer Slide Time: 47:32)



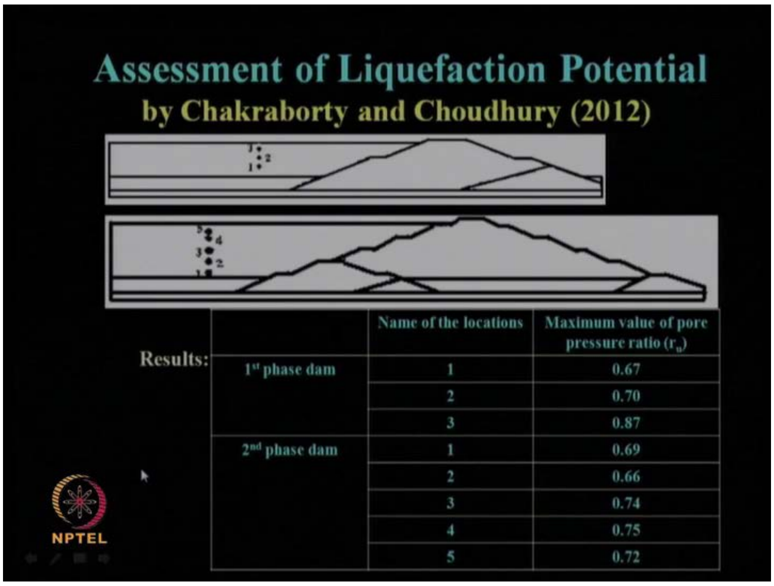
Whereas, for seismic analysis, as proposed by seed and Terzaghi for magnitude of about 6.4, these values are considered for analysis in TALREN and SLOPE W, because remember TALREN and SLOPE W, they cannot do direct slope stability analysis in terms of input acceleration motion, but FLAC can do that dynamic analysis. It will not give the value of factor of safety, it will give a displacement. You can recalculate back the value of factor of safety, in a wiser manner. So, these results shows different values of factor of safety, at different level if k h and k v, and corresponding yield acceleration value, which are much higher than the acceleration level, which is occurring at the site. It shows that, factor of safety will be always more than one, which is also getting proved from this results.

(Refer Slide Time: 48:41)



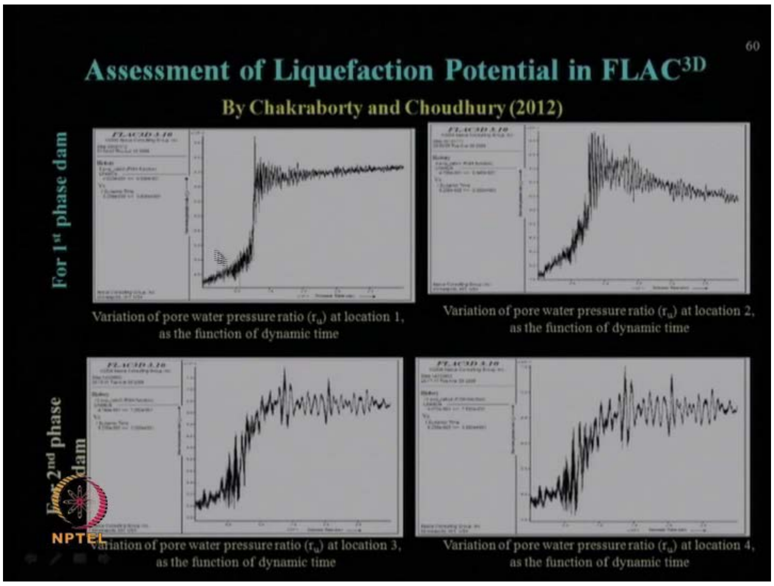
So, these are the contours in TALREN, you can find out. Now another additional thing we need to do for design of this tailing dam, what is that? We need to study the liquefaction analysis, liquefaction potential analysis. Like for any foundation we can do the liquefaction analysis, but for tailing dam, why it is additionally required, because in the upstream side you are storing some, or you are dumping some waste material, which initially is in the loose state. And also, most of the time it is dumped in the water, so that environmental hazards of spreading of those dust etcetera does not occur. So, there is a very high chance that, those loose dumped material, in your upstream side of tailing dam, may get liquefied if an earthquake comes; that is the reason why you need to carry out the liquefaction analysis for this tailing dam. Now how we have done for this project also. For this case study I will show here. This work is published by Chakraborty and Choudhury in 2012. This paper is available in the proceedings of second performance based design in earthquake geotechnical engineering, Taormina Italy conference. In this case in FLAC 3D, the liquefaction condition has been simulated using the Byrne model of 1991, using the SPT measurement of different layers and media, using the model Finn commend.

(Refer Slide Time: 50:22)



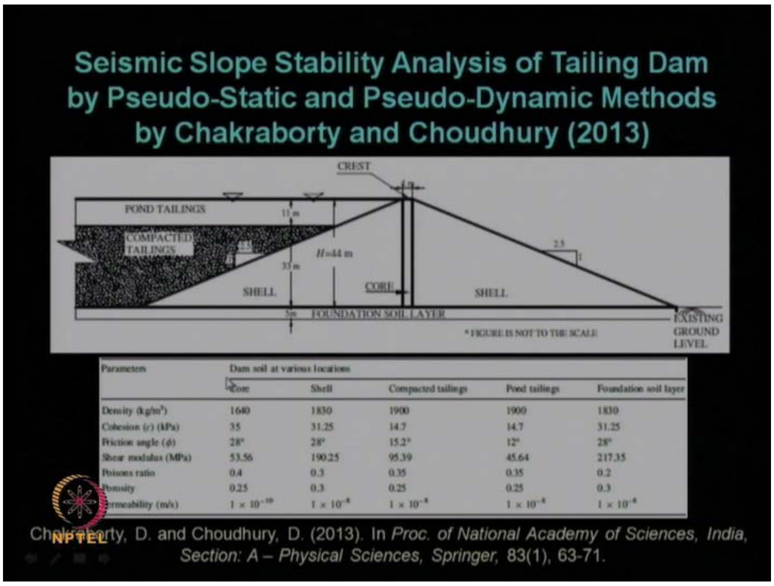
And these are the points at two different stages of the tailing dam, are concerned on the upstream side, as I said on the upstream side we are checking. So, this is for the first phase; one, two, three, at three different locations we are calculating the liquefaction potential. And for second phase, at this five different location; one, two, three, four, five, we are considering the liquefaction potential. How we are estimating it, in the FLAC, you will get the values in terms of the pore pressure ratio r u. Now r u value when it is equals to one means, it has been fully liquefied. If it is less than 1 then it is safe, but if it is close to one, there are chances, possible chances. So, like that we got, from our analysis for the chosen value of seismic acceleration at that zone, for first phase of dam, these are the values of r u. For second phase also these are the values, which are much lower than the one value. So, this portion is not going to get liquefied, under that zone two of IS code, as far the seismicity of that region is concerned, if that value of earthquake comes.

(Refer Slide Time: 51:38)



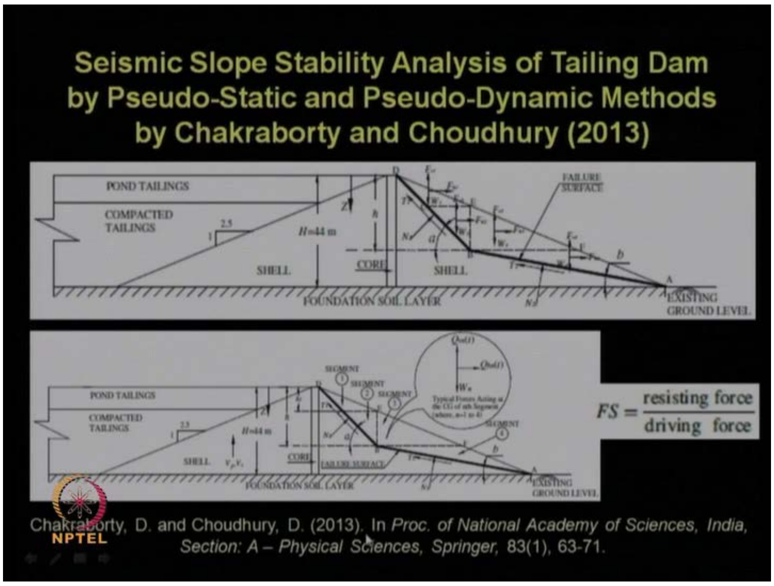
These are the output for liquefaction potential value in FLAC; you can see pore water pressure value at different location; location 1 and location 2 with respect to the dynamic time. So, whenever it tends to get saturated, you need to pickup that value, and similarly for the second phase of the dam also it can be obtained.

(Refer Slide Time: 52:00)



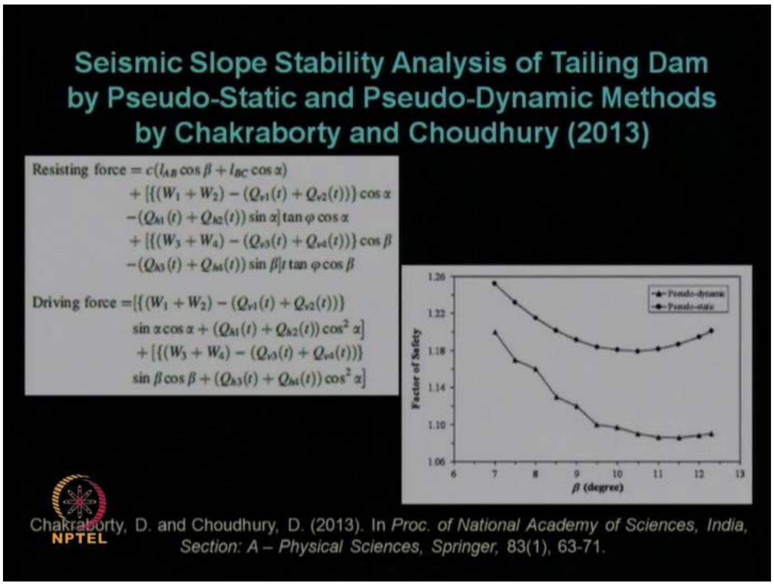
Now, next one comes, the seismic slope stability analysis, using pseudo-static and pseudo-dynamic method, because initially we have done using FLAC method and TALREN method software. Now we are using the analytical method, to check whether these slopes are stable or not. So, for that we have taken a sample problem like this, which is not the exact case as I have mentioned, we both studied sample case well as exact case. This is available in the journal paper by Chakraborty and Choudhury 2013 this year. In the journal proceedings of national academy of sciences India, section A, physical sciences, Springer publications, Springer journal. This is the volume number of the journal, page number. So, this is the cross section of the tailing dam, and other input data of the parameters.

(Refer Slide Time: 52:53)



This is the downstream side failure surface are considered, based on the frenetic line etcetera, and factor of safety can been calculated like this as we know. In this case, the upper figure shows the all the forces, including the seismic inertia force in vertical and, in horizontal and vertical directions, considering pseudo-static approach. And this one shows the same problem, but these inertia forces are considered or calculated using Pseudo-dynamic approach.

(Refer Slide Time: 53:25)



So, with that, the expressions for resisting force and driving force can be calculated. We have calculated like this, and finally the factor of safety has been obtained for different values of this beta angle. You can see, here factor of safety with respect to pseudo-dynamic, is much lower than what we got and pseudo-static. It is a case specific as I said. It is not always true; it can interchange also, but this case we got pseudo-static is giving higher value of factor than the pseudo-dynamic. So, always we need to check which one is giving more critical value, and why it has happened, let me tell you, because in pseudo-dynamic we have considered the, possible amplification, which we obtain form the side data. So, that chance or that option of considering the soil amplification is not there in your pseudo-static analysis. So, using that concept, one can easily analyze any tailing dam like this. So, with this we have come to the end of today’s lecture, we will continue further in our next lecture.