Geotechnical Earthquake Engineering

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Module - 9

Lecture - 38

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. So, for this video course, currently we are going through module number nine, which is seismic analysis and design of various geotechnical structures. Within this module nine, a quick recap what we have learnt in our previous lecture. In the previous lecture, we discussed about seismic design of waterfront retaining wall or sea wall.

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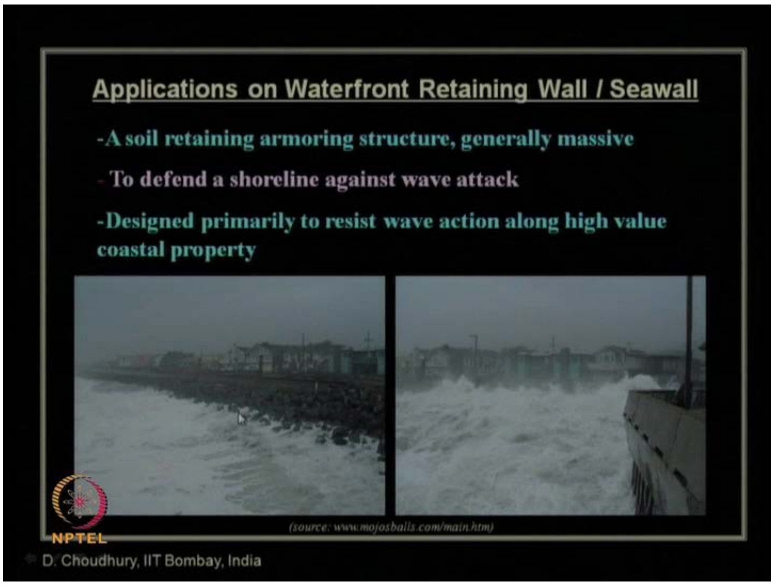

So, what we have learnt that generally this kind of waterfront structures or waterfront retaining wall, which are provided to protect the shore and the properties from the sea. Those are nothing but generally massive structure to defend a shore line against wave attack, and design primarily to resist wave action, along the high value of costal property, like we have in Mumbai also in marine drive.

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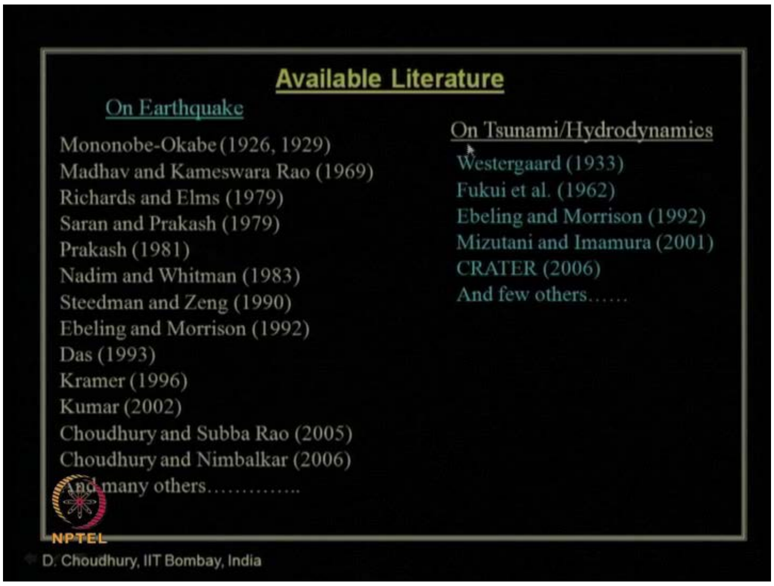

We have also seen that what are the available literature on the individual work, on the area of earthquake engineering for retaining wall design, and on the tsunami and hydrodynamics effect on this waterfront retaining structures, by these researches. But none of them had considered the combined effect of this earthquake and tsunami together, because sighting that both are extreme events. It is highly not so possible that both the events are occurring together. However, in the very recent 2011 march, Tohoku earthquake in Japan, entire world had experienced that the, from the experience of Japanese in Tohoku region, that along with tsunami even the post-earthquake or aftershocks of considerable magnitude can come, which need to be considered. So, that is why it is very important to study the combined effect of this earthquake, along with the effect of tsunami on this waterfront structures or retaining walls.

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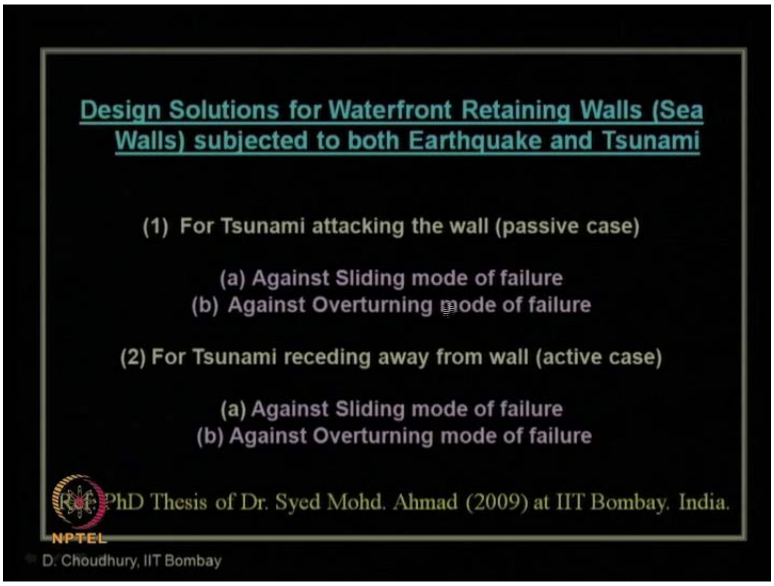

So, for this sea walls, design, what are the basic two things needs to be considered, as I have already mentioned this is the PhD thesis work of my second PhD student Dr. Syed Mohammad Ahmad, who completed his PhD in 2009 at IIT Bombay, and currently he is a lecturer at University of Manchester in UK. He studied, for his PhD program under my supervision at IIT Bombay that the for design of waterfront retaining wall or sea wall, basically two important aspects, or two important cases will arise. One case is when tsunami is attacking the wall, and another case is when tsunami is receding away from the wall, or going back to the sea. So, within both of the cases, the major two aspects are; one is sliding mode of failure, and another is over turning mode of failure, both in this case, as well as in this case. The first case can be referred to as passive state of earth pressure.

Whereas the second state can be considered as active state of earth pressure, based on the movement of the wall towards the back fill soil or towards the shore. So, as we can see here, when a wall is standing like this, and suppose this side we have wave; that is water side, and this side we have shore or the ground side with back fill soil. So, when the tsunami wave is hitting this wall, wall tends to move towards this soil. So, it is a passive state of earth pressure; that is tsunami attacking the wall, and after it over tops the wall, after sometime through weep hole etcetera, the water goes back to the sea. So, the tsunami always receives back and goes back after sometime, to the sea. That time it drags the wall towards the sea side; that is, it moves away from the backfill soil which is nothing but active state of earth pressure. So, both these state of earth pressures are considered, along with the tsunami wave pressure and earthquake forces acting on the wall.

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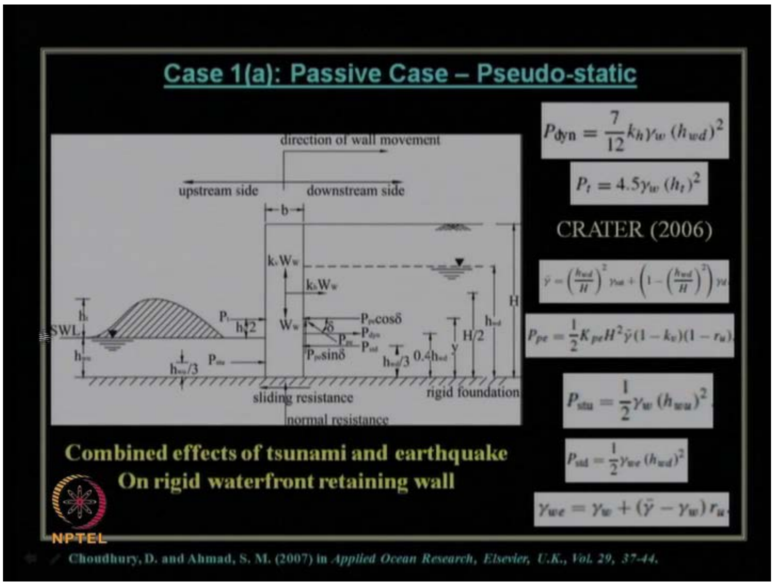

Then we had seen, this basic diagram, how the model has established by Dr. Ahmad. So, this can be found out in the detail in Journal Choudhury and Ahmad 2007, in Journal applied ocean research published by Elsevier. This is the volume number and page number. This case is for the passive case, using Pseudo-Static Approach of earthquake loading. So, this is the direction of wall movement, which is nothing but the passive state. This side is the shore line or ground surface with backfill soil, at a certain height there is a water table, where as this side is upstream side where there is water. So, this is the height above the still water level, which is nothing but the height of the tsunami.

And this is tsunami wave pressure, then this is hydrostatic water pressure. These are the inertia forces acting on the wall, and this is weight of the wall, along with this is the passive earth pressure acting from the soil site. So, considering the equation of hydrostatic pressure or pressure force, given by waster guard, this equation p hydrodynamic, and the tsunami wave pressure force, the equation as proposed by crater, and considering the variable other parameters like average unit weight in the downstream. The passive earth pressure on this side has been estimated using this equation, and also from the downstream side, the static pressure and from the upstream side, the hydrostatic pressures are obtained using this relationship.

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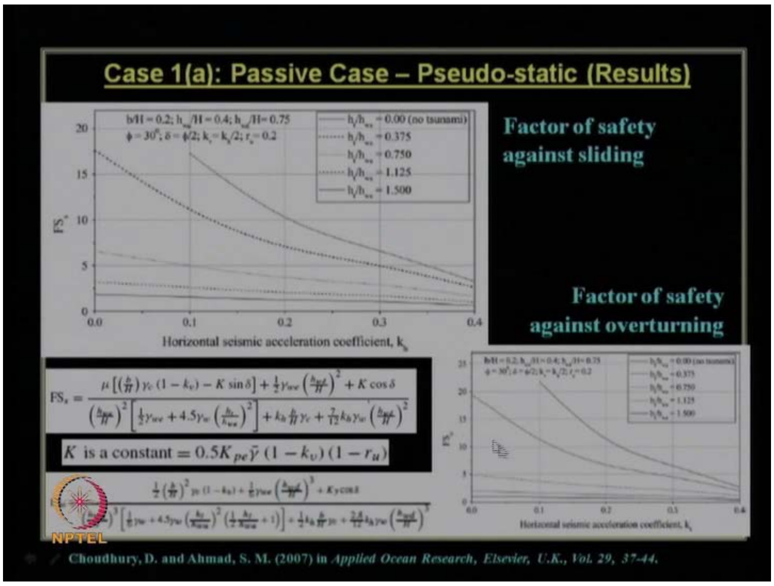

Then, we had seen, that for the passive state, these are the results which shows the factor of safety against, sliding mode of movement with respect to various seismic horizontal acceleration coefficients K h value, and for various height of tsunami height of the water, compared to the static height of the water in still water level. So, when it is 0, then; that means, there is not tsunami, but when the tsunami starts coming, there can be variable height. So, based on that, we found that factor of safety against sliding, for this waterfront retaining wall or sea wall is significantly decreasing, as the seismicity is increasing, like with increase in K h, as well as the tsunami wave height is increasing. So, the combined effect can easily be obtained from this proposed design chart by us.

Suppose at K h value of 0.2 g, there is a height of the tsunami wave height; say 1.125. Then we have to go to this line, at this point, this will give us the factor of safety of the wall, against sliding for a chosen other input value as shown over here. So, if it not satisfying the stability criteria of 1.15, minimum factor of safety against sliding under earthquake condition, then we have to redesign this wall section to withstand both seismic acceleration, as well as the tsunami wave height at a particular region. And this equation gives us the closed form solution, factor of safety against sliding, using which one can easily design the cross section of a retaining wall to withstand, certain value of seismicity coefficient, as well as the tsunami wave height. Similarly for factor of safety against overturning mode of failure also, the closed form solution is given over here. And the variation of that factor of safety, with increase in K h and with increase in tsunami wave height, is shown over here.

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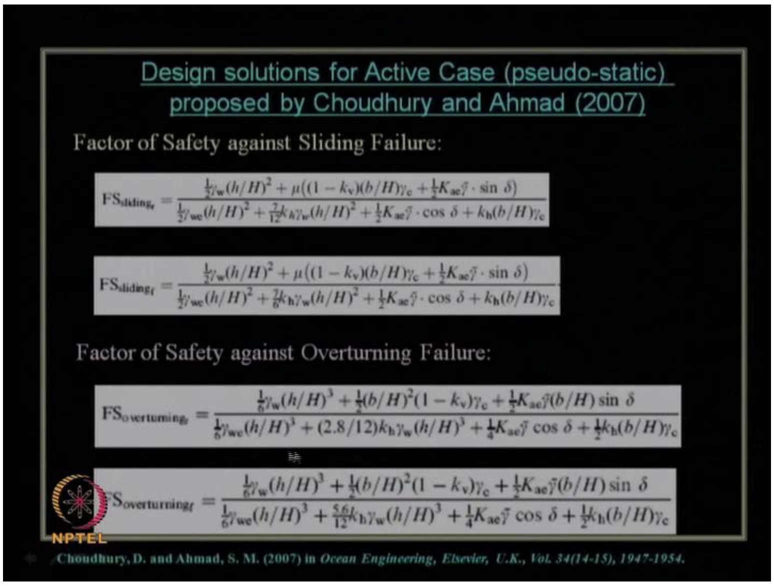

Another case that is design solution for the active state of earth pressure; that is when tsunami wave is going back to the sea, using the pseudo static approach for earthquake loading. It has been proposed the solution a design, solutions are proposed by Choudhury and Ahmad in 2007. This Journal paper Choudhury Ahmad 2007, in the Journal Ocean Engineering, published by Elsevier. This is the volume number and page number. You can see the factor of safety, against sliding mode of failure, when we are considering this mode that is, when tsunami wave is going back to the sea; that is receding back. In that case for each of this mode of failure; that is sliding and over turning, we have two different conditions, what are those two different conditions. As we have discussed in our previous lecture, it is based on the permeability of the back fill soil, and also how that drainage through the wall using wipe hole etcetera or filter design etcetera, has been provided.

So, based on that one condition can be, free water movement; that is when the permeability of the backfill material is good, as well as there is a free drainage provide in the walls section. So, that no water stands in the back fill soil, when tsunami is going back to the sea. But there is a possibility or a case, when the permeability of the soil is not very high also, the design of filter drains, and wipe holes are not properly designed. In that case it should be considered as restrain water case in the back fill. So, considering these two criteria, as I have said, factor of safety against sliding, small r denotes restrain water condition, and small f denotes free water movement condition. So, under both these conditions we can have, the closed form solution of factor of safety against sliding, similarly for factor of safety against overturning, retrain water case, as well as free water case are proposed by us, by given this equations.

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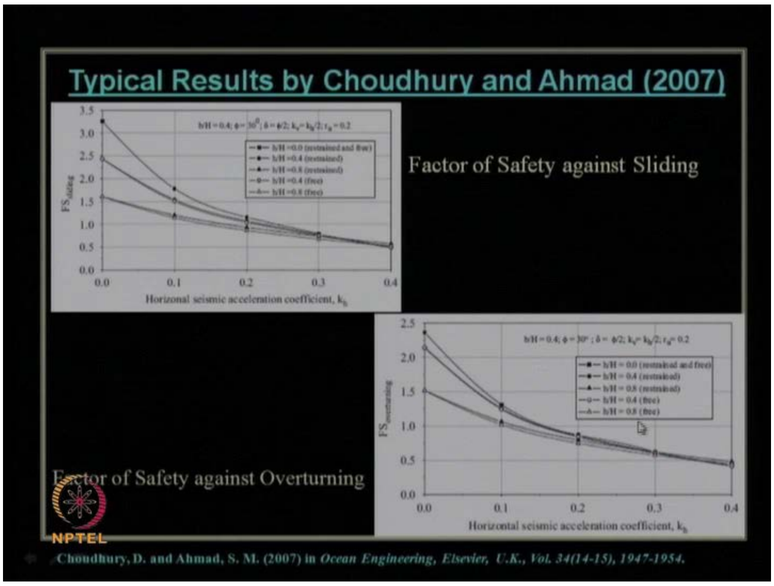

And the results, shows typically over here; like factor of safety against sliding with respect to K h value, as we can see K h value increases, factor of safety drastically decreases for this chosen input value. And the variation with respect to the restrain water or free water case also can be seen over here. So, in this case as we know tsunami wave height does not matter, because tsunami is residing back or going back to the sea. Similarly factor of safety against overturning also dependent on the values of K h K v, and other input parameters like restrain water height condition, and free water height condition.

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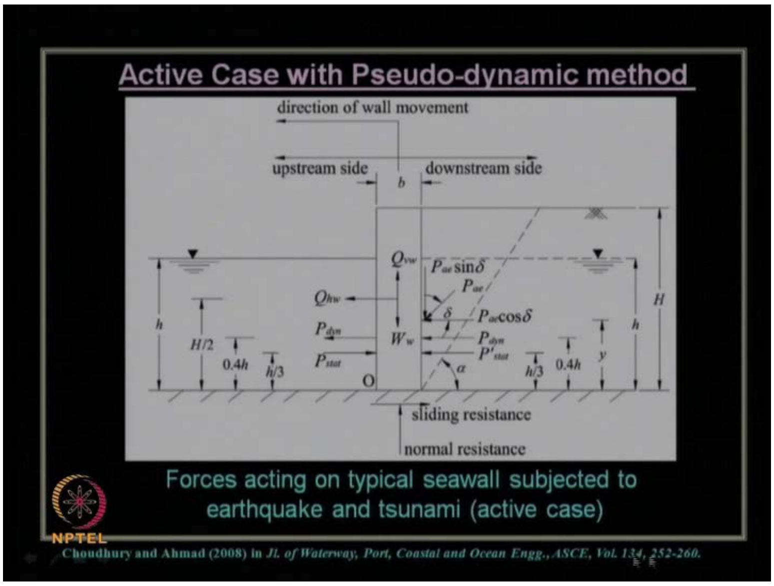

Then we had discussed in our previous lecture, also about this active state of earth pressure, for waterfront retaining wall using pseudo-dynamic approach of earthquake loading. So, for pseudo-dynamic force, this is the direction of wall movement, because it is the active state, tsunami is going back to the sea. The details are available in this Journal paper Choudhury and Ahmad 2008, published in the Journal of Water Wave Port, Costal and Ocean Engineering of ASCE, USA. This is the volume number and page numbers.

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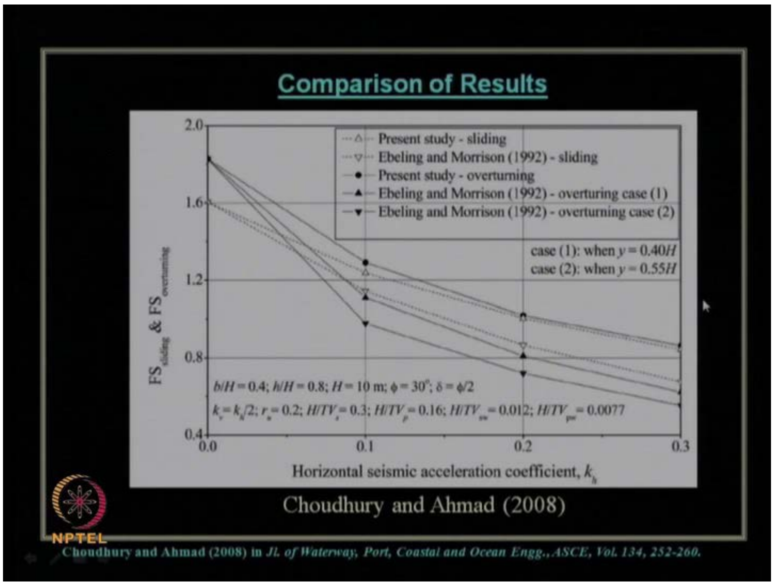

And the results of factor of safety against sliding and over turning with respect to K h values are shown over here, and compared this present study results with the available results in the literature, as proposed by Ebeling and Morrison in 1992 which is, used by U S army corps of engineers, for design of sea wall or waterfront retaining wall. And you can see the present study gives the critical solution compared to the pseudo-static result, because pseudo-dynamic results, considered the dynamic effects of this earthquake forces and tsunami forces effectively, compared to pseudo-static result.

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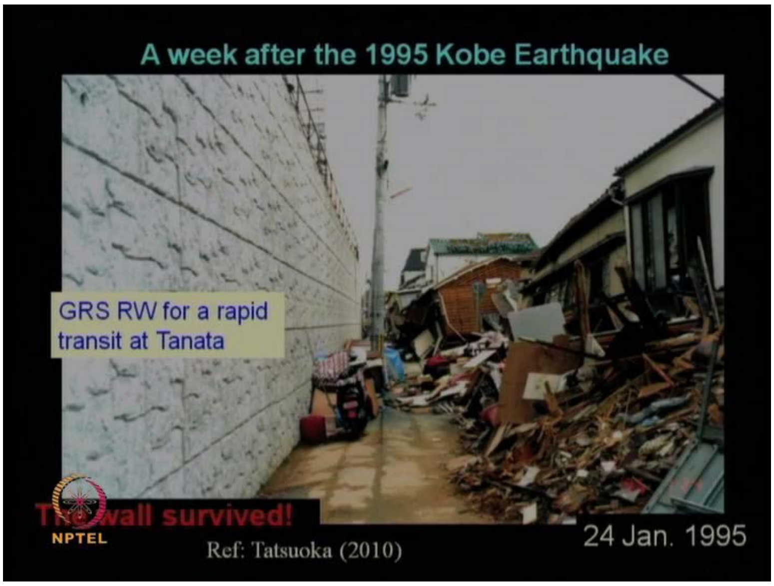

Next we had also discussed the seismic design of reinforced soil wall in our previous lecture. We also mentioned as given by Tatsuoka in 2010; that during and after 1994 Kobe earthquake in Japan it was found that, geo-synthetic reinforced soil wall, they survived that big damaging earthquake of 1994 of Kobe. Whereas, the conventional structures, or conventional buildings and walls. They could not sustain that that, or could not survive that huge magnitude of earthquake of 1995 Kobe. It automatically shows the, application of geo-synthetic reinforce soil wall, which can withstand, more earthquake loading compared to, conventional retaining wall without reinforcement.

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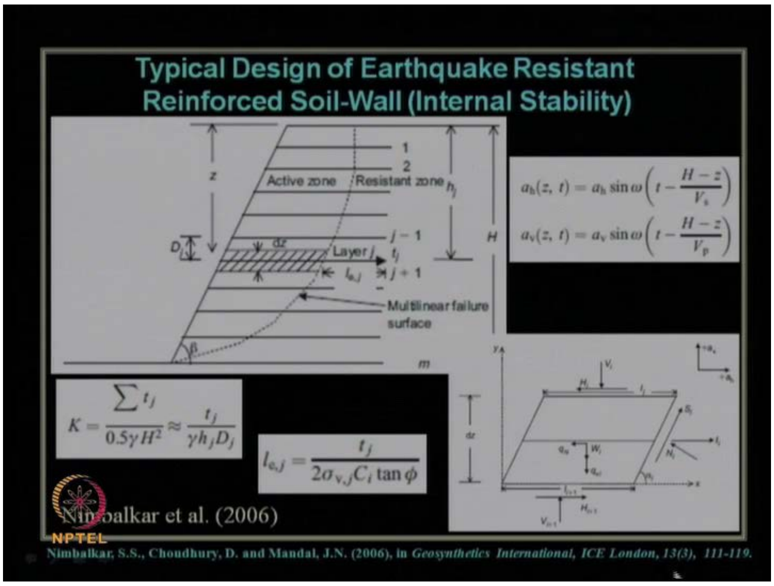

Then we talked about the analysis for reinforce soil wall, to make it earthquake resistant design, considering the internal stability criteria, because there are two stability criteria for reinforce soil wall; one is internal stability, another is external stability as we have discussed in previous lecture. Internal stability looks into the aspect of failure of this reinforcement within the soil, in terms of pull out, in terms of strength of the material. So, this is the reinforce soil zone, in which each layer, through each layer there is infinitesimal small element was considered, and the forces acting on that element is shown over here, through the analysis using pseudo-dynamic approach, for by considering the seismic horizontal and vertical accelerations, as given by this equations.

Finally, the strength of the reinforcement required to withstand a particular magnitude of earthquake, is proposed by this non dimensional parameter K, and t j is the strength of that reinforcement of j eth layer. And the length of that layer to be provided, for that is pullout resistance stability, is expressed by this expression. This analysis is available in the Journal paper of my first PhD student Nimbalkar et al 2006, Nimbalkar, Choudhury and Mandal in 2006, published in the Journal Geo-Synthetics International, published by Institute of Civil Engineers London UK. This is the volume number and page number.

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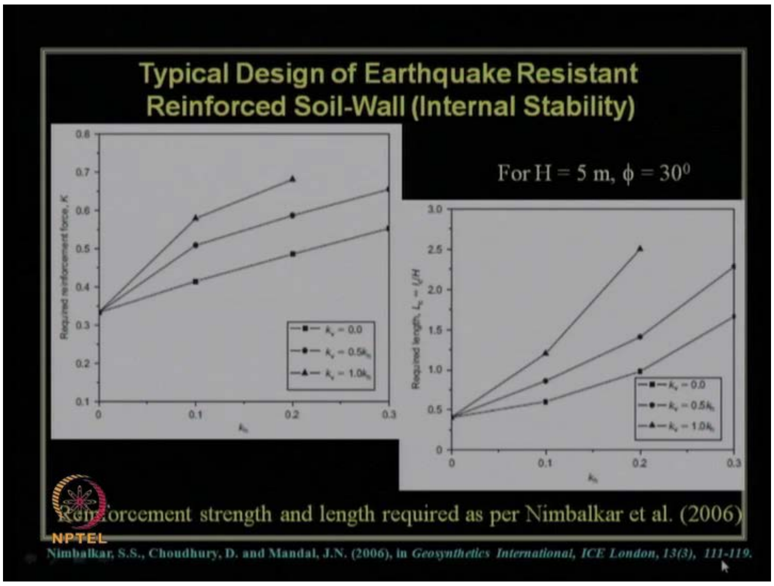

We have seen the results that finally, the design charts were proposed, like how much reinforcement force is required for different values of K h and K v. So, using this design chart one can easily estimate, the amount of reinforcement strength required, as well as how much is the length of that reinforcement is required to be provided, against pullout failure of this reinforcement for, a particular value of K h and K v thorough this proposed design charts.

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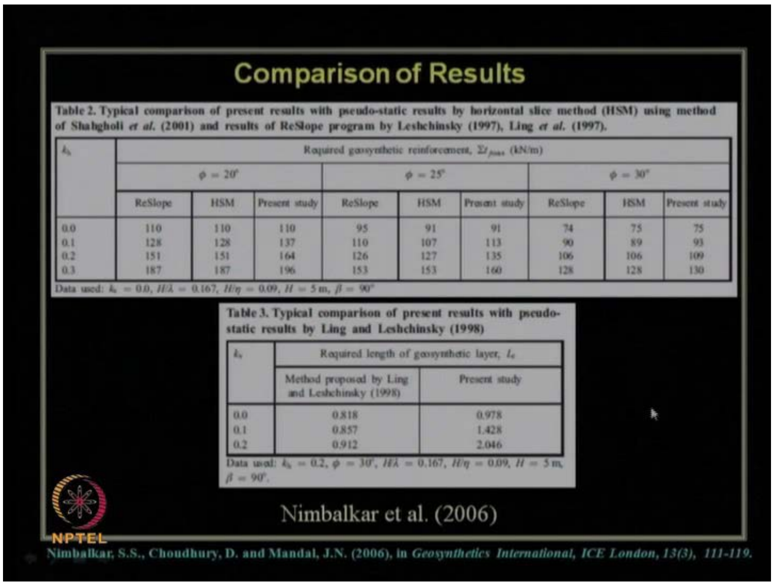

Then we had compared our results, the pseudo-dynamic results of this geo-synthetic reinforcement strength, as well as the length required with respect to, the previous researchers result who used the pseudo static approach; like Ling and Leshehinsky results and Ling et al 1997 results and Shahgholi et al 2001 results, for various values of seismic acceleration coefficient. We can find here that, the present study of using pseudo-dynamic approach, always gives a critical design, which automatically shows that, importance of using the pseudo-dynamic approach compared to conventional Pseudo-Static Approach.

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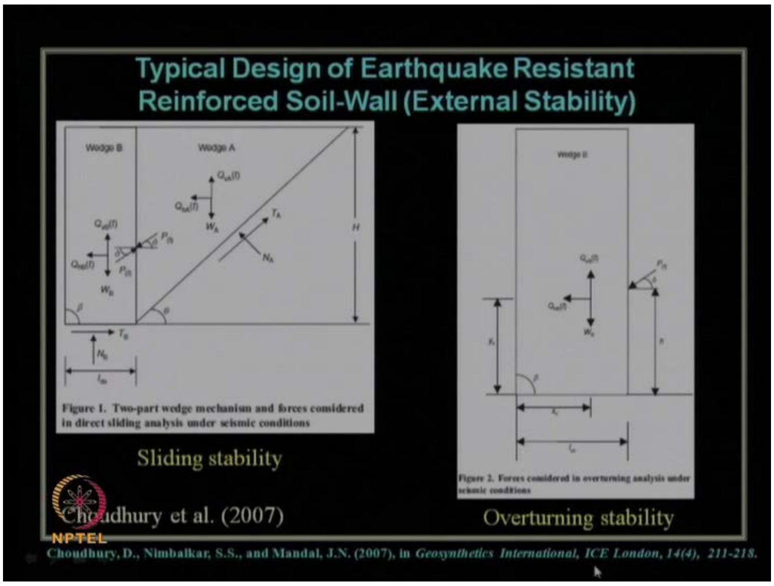

Then we had also discussed in the previous lecture about the external stability criteria of this reinforced soil wall, where the reinforce soil zone has been sub divided into two portions; one is wedge A, two zones wedge A triangular wedge, and wedge B is rectangular wedge, both sliding as well as overturning stability, in terms of this external stability criteria was considered. And the entire analysis can be obtained in the Journal paper Choudhury et al 2007, Choudhury, Nimbalkar and Mandal 2007, which is published in geo-synthetics international Journal, published by institute of civil engineers London, this is the volume number and page number.

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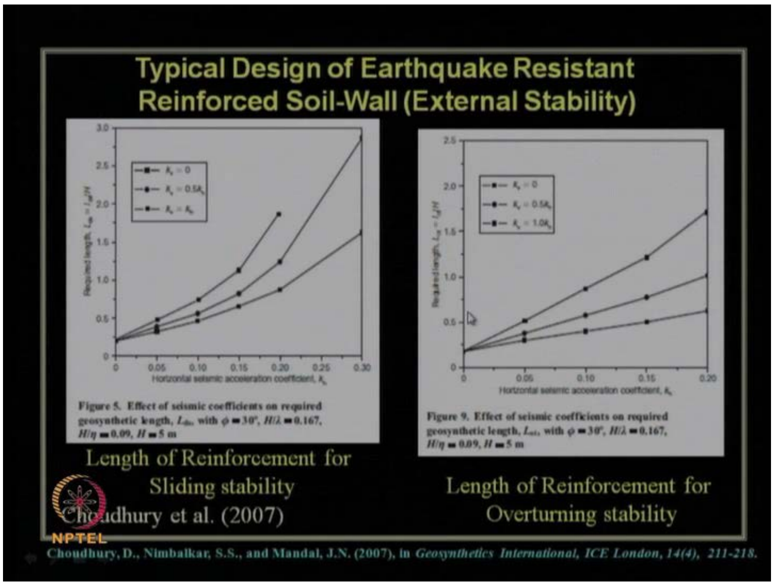

And finally, the results are shown, in terms of required length of reinforcement for direct sliding stability, as well as over turning stability, for a particular value of K h and K v. So, when somebody is designing this reinforce soil wall, they need to provide the reinforcement strength as per the internal stability criteria as we have discussed. And the length to be provided in terms of, either external stability, both sliding and over turning, and considering internal stability of pullout criteria among these three, whichever gives maximum length that needs to be provided for this earthquake resistant design of reinforcement soil wall.

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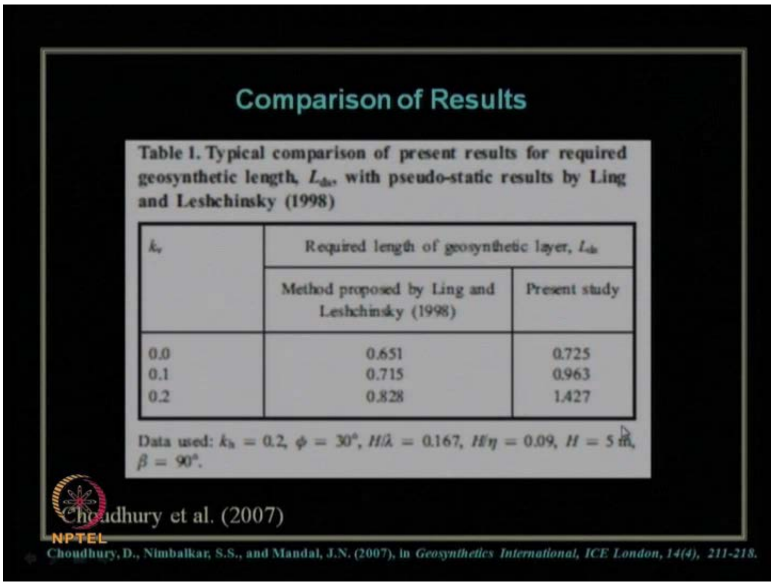

Table 1. Typical comparison of present results for required geosynthetic length, L_{ds}, with pseudo-static results by Ling and Leshchinsky (1998)

k_v	Required length of geosynthetic layer, L_{ds}	
	Method proposed by Ling and Leshchinsky (1998)	Present study
0.0	0.651	0.725
0.1	0.715	0.963
0.2	0.828	1.427

Data used: $k_h = 0.2$, $\phi = 30°$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m, $\beta = 90°$.

Then we had compared our results of pseudo-dynamic approach with respect to Ling and Leshehinsky's 1998 results, in terms of non dimensional length to be provided to withstand the direct sliding for different seismic acceleration, ass you proposed by using Pseudo-Static Approach. And one can find out that pseudo-dynamic approach gives the most critical results. So, with that we had completed our previous lecture.

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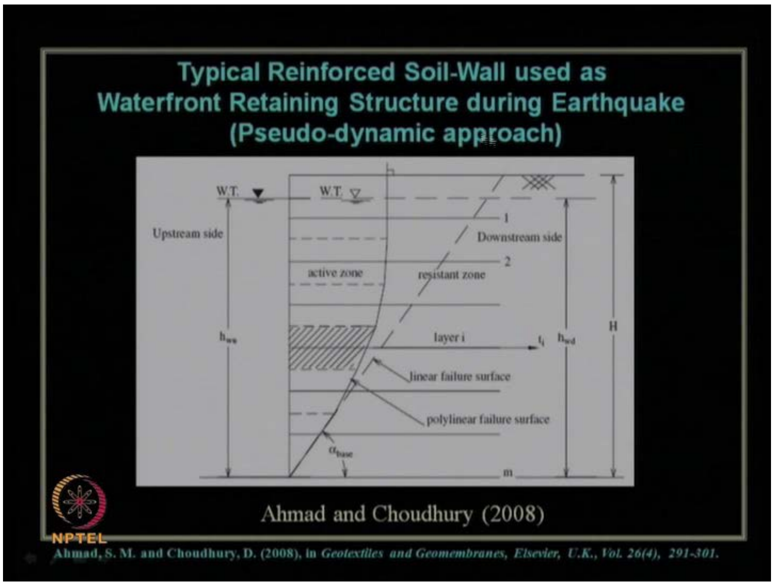

So, in today's lecture, we will start with the seismic design of waterfront reinforced soil wall. In waterfront reinforce soil wall also, the similar method has been adopted using pseudo-dynamic approach. This is the waterfront reinforce soil zone, this is the upstream side. And in this case both linear as well as poly-linear failure surfaces were considered for the analysis by Dr. Syed Ahmad, under my supervision at IIT Bombay for his PhD thesis. So, these details are available in the Journal paper Ahmad and Choudhury 2008, in Geo-Textile and Geo-Membranes Elsevier publication, this is the volume number and page number. This linear failure surface is just for the sake of academic interest, but as we know from the experience that, mostly at side the poly-linear failure surface like this will get formed. So, we have provided the results and compared the results for both the cases, considering also the hydrodynamic pressure, as it is acting for the case of waterfront retaining structures, using this reinforce soil wall concept.

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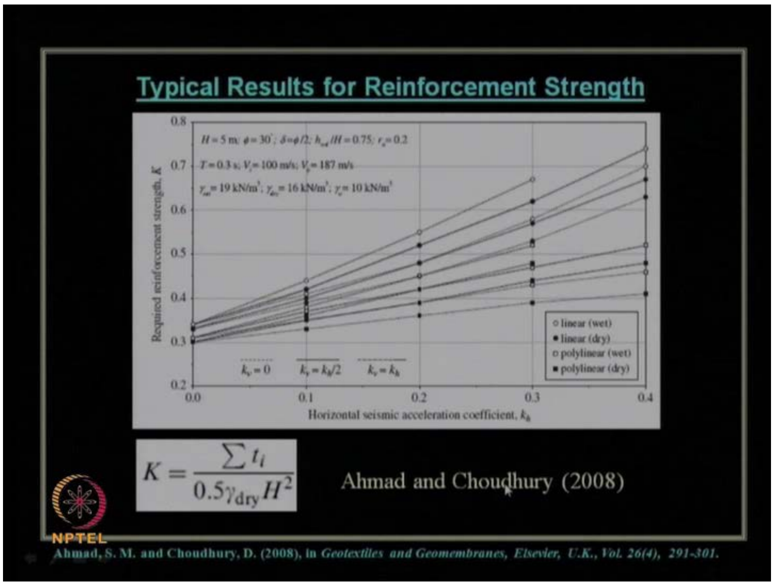

So, this is the final design chart here again, how much reinforcement strength is necessary, to withstand the hydrodynamic force and also, this earthquake force of K H and K v different magnitude. One can use this design chart to get this, non dimensional parameter of reinforcement strength which is necessary to be provided for stability.

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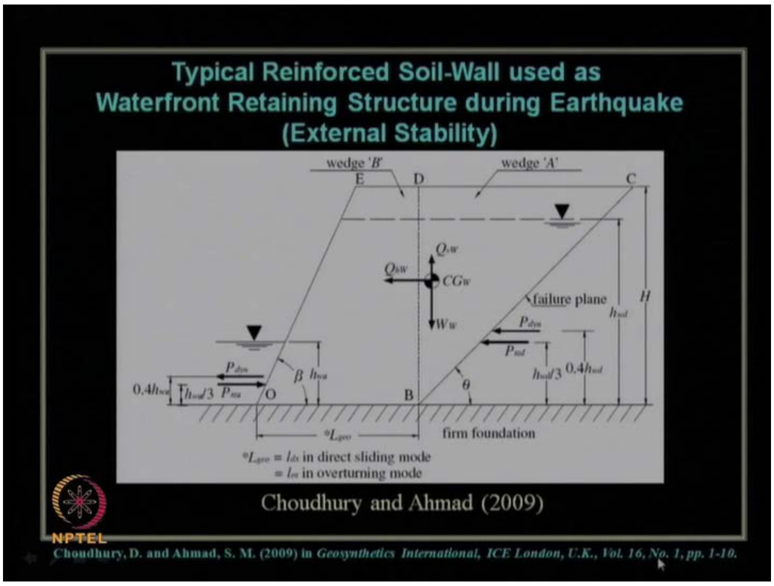

Also the external stability criteria for reinforced soil wall used as waterfront retaining structure was considered, using two wedge mechanism; wedge A and wedge B. Considering both, direct sliding mode of failure as well as the over turning mode of failure, and the details of this external stability analysis are available in the Journal paper by Choudhury and Ahmad 2009, in published in the Journal Geo-Synthetics International, Institute of Civil Engineers London U K, this is the volume number and page number.

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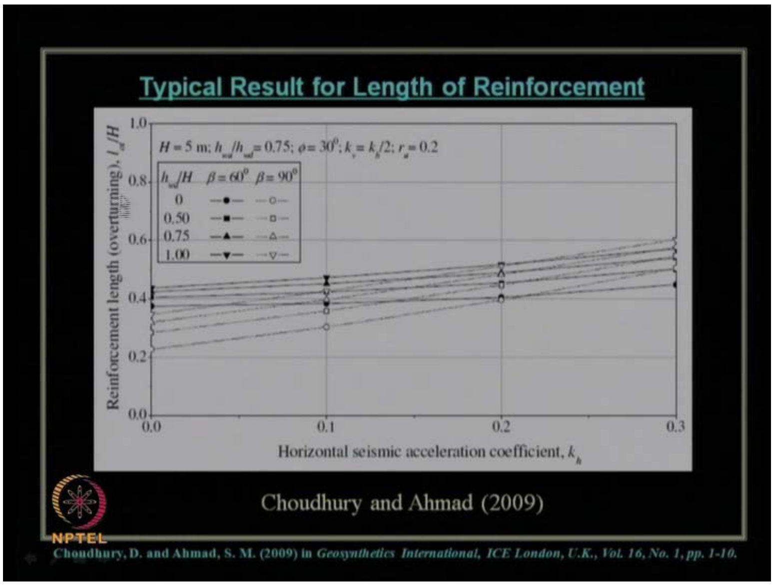

And the results here again the design charts have been proposed, to calculate how much reinforcement length is required, in the non dimensional form, the design charts are provided. So, that designers or practitioners can use it very effectively and easily, for different values of input seismic acceleration, how much value needs to be provided for overturning mode of failure. Similarly for sliding mode also the values have been provided.

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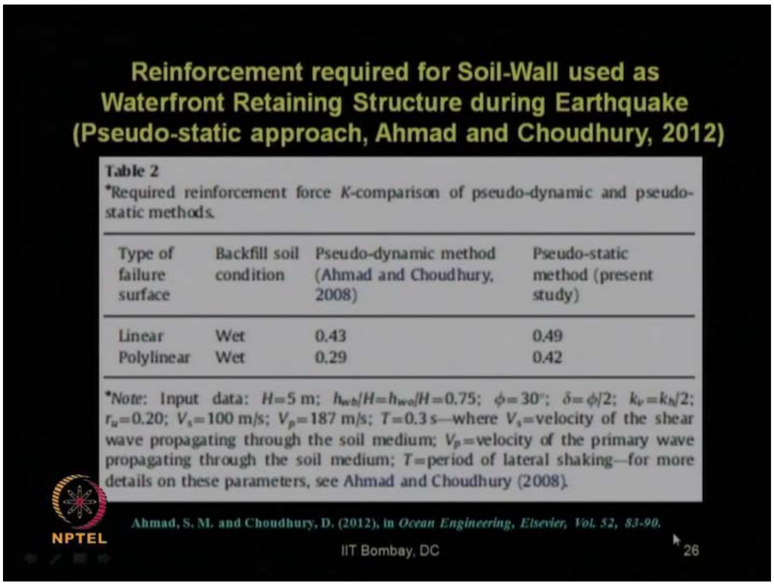

Then, this table shows the comparison of pseudo static approach results, as given by Ahmad and Choudhury in 2012, compared to pseudo-dynamic results of Ahmad and Choudhury 2008, which we have mentioned just now. You can see over here, that the value of this K; that is the reinforcement strength which is required. As far as pseudo-dynamic approach and pseudo-static method is concerned, you can see pseudo-static method in this case is giving higher value. So, it not always necessary that pseudo-dynamic will give the lowest possible value. It can depend on the various input parameters etcetera. So, for the waterfront retaining wall reinforce soil case, we found that pseudo-static results are giving the optimum value, or the design value, or critical value of results.

So, these details are available in the Journal paper Ahmad and Choudhury 2012, published in Journal Ocean Engineering, published by Elsevier, this is the volume number and page number. Now, let us come to next subtopic; that is seismic design of shallow footings. Now shallow footings or shallow foundations we use extensively, for several structures, like small buildings, like one story or two story building at various places. So, how to make these foundations earthquake resistant, or how to design this foundation to withstand certain magnitude of earthquake, so that there is no damage. This topic will mention us how to design those shallow footings or shallow foundations, which can withstand certain magnitude of earthquake, depending on the soil conditions.

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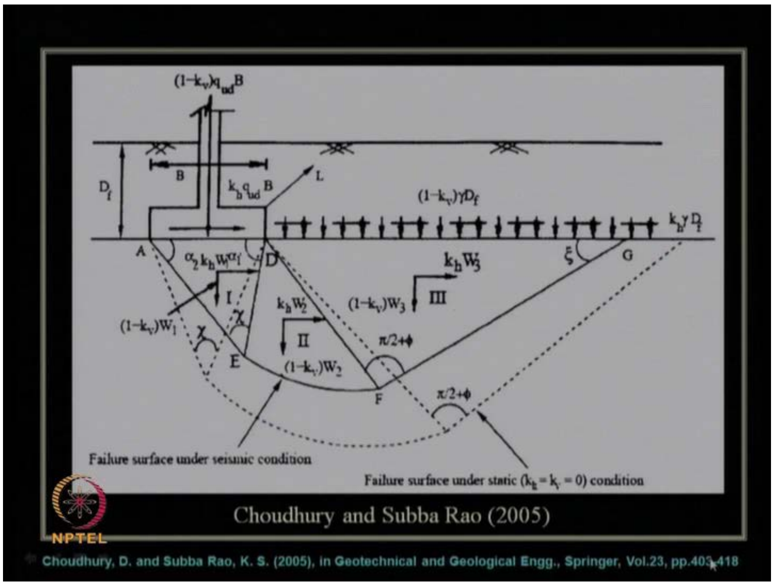

So, this is the analysis, you can see over here this is the line diagram of a cross section of shallow isolated strip footing. This is the width of the footing B. This is the depth of embedment of the shallow footing D f. And as per Terzaghi's definition, if it is shallow footing then, this ratio of D f by B should be less than or equals to 1. And this is the length of the footing, this length has been considered, much larger than this width of the footing, so that we consider as a strip footing for the analysis. As Terzaghi did for the static case of bearing capacity factor determination, this dotted line shows the typical failure surface as for Terzaghi's failure mechanism is concerned.

Now this dotted line is actually symmetric under static condition. So, whatever we see on this side, the similar thing we can find on the other side, under the static loading condition. However, it is not the case in case of the seismic loading; why, because at seismic loading, when we are considering the pseudo static approach for the seismic loading. Suppose K h is the horizontal seismic acceleration, and K v is the vertical seismic acceleration. The net effective load in the vertical direction will be 1 plus minus K v, q u d is nothing but ultimate bearing capacity of the soil, times this b, width of the footing. Now, this q u d we need to estimate as for as Terzaghi's method, in static case is considered we need to find out in dynamic case, for the proposed solution, as we are detailing over here.

And K v is vertical seismic acceleration, it can act in both upward as well as downward, and we need to consider the critical direction, which gives the minimum value of this bearing capacity, so that it gives us, a one sided failure mechanism, because if it acts in this direction; obviously, the other side, the full passive earth pressure is not going to get developed, because of that it will not develop a failure surface in the other direction, but it will develop failure surface only in this direction. So, it will be one sided failure mechanism, like this, and these are the three typical zones under the seismic condition. The details of this can be obtained, in the Journal paper published by Choudhury and Subba Rao. This is a part of my PhD thesis work, under the supervision of Prof. K S Subba Rao at IISC Bangalore. This paper Choudhury Subba Rao 2005 is available in Journal paper Geotechnical and Geological Engineering, published by Springer; this is the volume number and page number.

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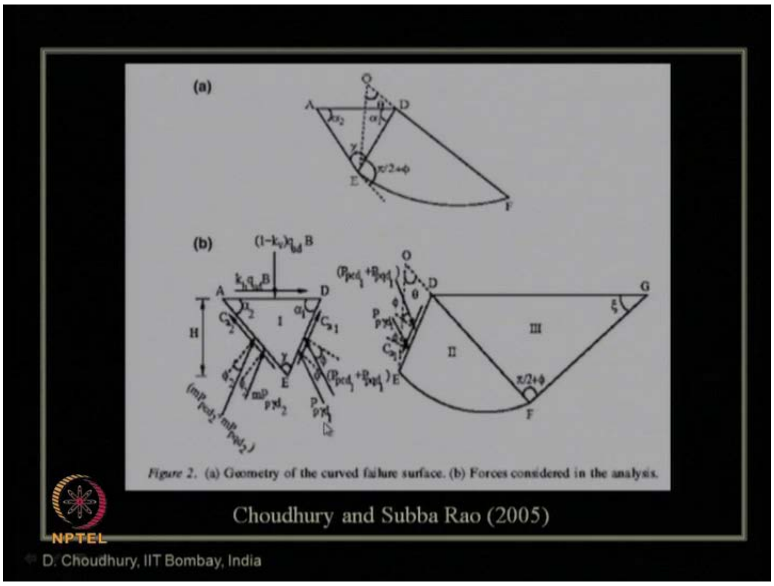

So, the forces are acting on these three different zones. So, this is the failure mechanism; this is zone 1, zone 2, and 3 is nothing but we can imagine this D E as a imaginary retaining wall as was considered by Tarzagi for static case also, but in this case it will be all the dynamic forces. And this exit angle will no longer be a static angle, but the dynamics exit angle. And this, what are the important changes with respect to static, in static these two angles are equal, as we know for Terzaghi’s analysis with rough footing, this is phi value. Whereas in this case these are non equal alpha 1 and alpha 2. Alpha 1 will be more than phi and alpha 2 should be less than phi. So, we have to find out what should be the value of these two angles, corresponding to this will be full phi; that is one sided failure mechanism. So, this angle of internal friction between this zone and this zone, which is nothing but phi soil friction angle, but in this side where full passive pressure is not getting developed, this m factor is a factor which is less than one. It will be equal to one under the static case, when both the sides are getting formed with the same failure mechanism. So, this phi 2 value; that is the mobilized interface soil friction angle, will be lesser than this value of phi.

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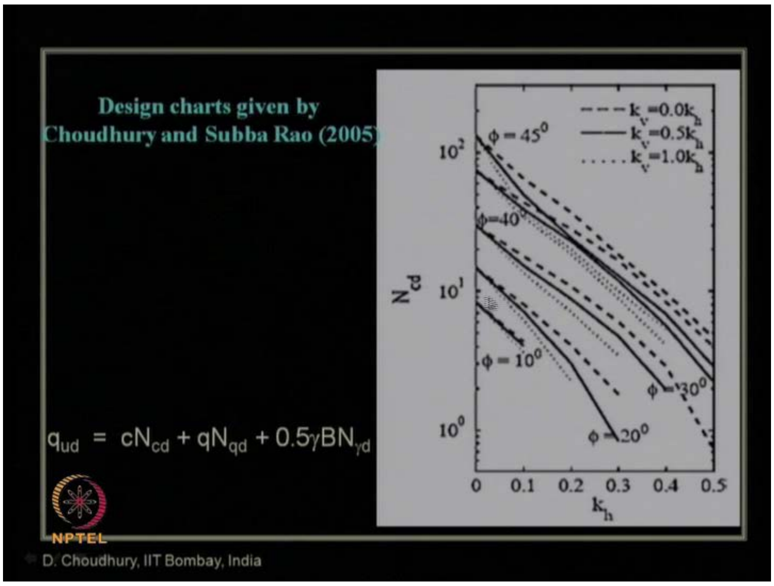

So, we have to do a iteration of these all parameters involved in the analysis, and finally we got the design charts, which are proposed in terms of the bearing capacity factors under dynamic condition. And this is the proposed equation which can be used to calculate the dynamic bearing capacity, q u d equals to c N c d plus q N q d plus half gamma B and gamma d. And this is extension of Terzaghi's theory from static case, but d indicates the dynamic cases. So, N c d is the dynamic bearing capacity factor, in terms of cohesion. N q d is dynamic bearing capacity factor in terms of surcharge. And N gamma d is dynamic capacity factor in terms of unit weight. We can see that as K h value and K b value increases for different values of soil frication angle phi. There is a significant decrease of this bearing capacity factor. So, at K h equals to 0 and K v 0, these values are nothing but the static bearing capacity factors, which are equal to the Terzaghi's bearing capacity factor, as we are extended Terzaghi's method. But under dynamic condition our present study shows, the critical decrease of this baring capacity factor, which need to be considered at a particular site, knowing what is the value of input value of K h, and for particular value of phi, then for a chosen K v value for the design, we can get what is the N c d value.

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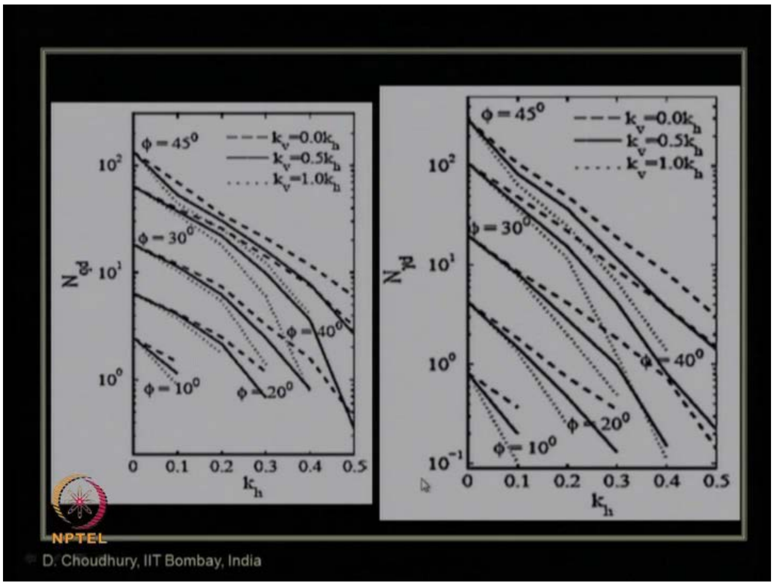

Similarly, for N q d and n gamma d also this design factors can be obtained. And finally the design of this shallow, isolated footing can be done, using this proposed design charts.

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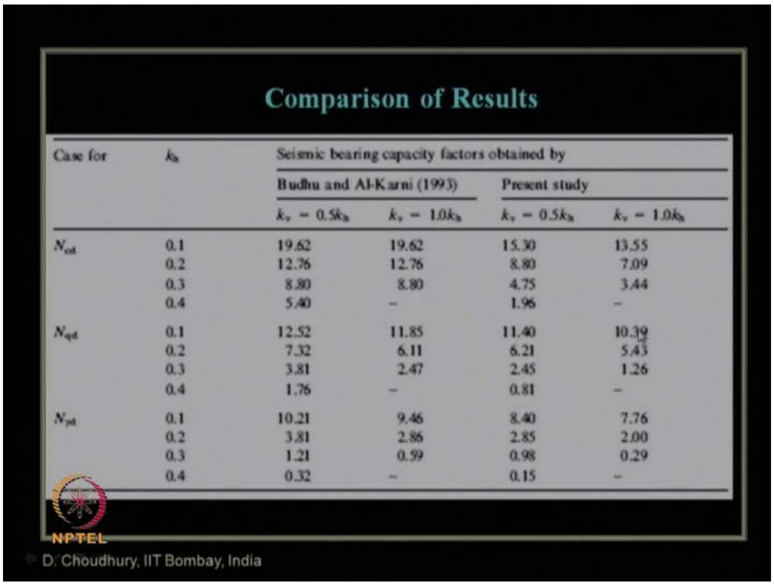

Comparison of Results

Case for	k_h	Seismic bearing capacity factors obtained by			
		Budhu and Al-Karni (1993)		Present study	
		$k_v = 0.5k_h$	$k_v = 1.0k_h$	$k_v = 0.5k_h$	$k_v = 1.0k_h$
N_{cd}	0.1	19.62	19.62	15.30	13.55
	0.2	12.76	12.76	8.80	7.09
	0.3	8.80	8.80	4.75	3.44
	0.4	5.40	–	1.96	–
N_{qd}	0.1	12.52	11.85	11.40	10.39
	0.2	7.32	6.11	6.21	5.43
	0.3	3.81	2.47	2.45	1.26
	0.4	1.76	–	0.81	–
$N_{\gamma d}$	0.1	10.21	9.46	8.40	7.76
	0.2	3.81	2.86	2.85	2.00
	0.3	1.21	0.59	0.98	0.29
	0.4	0.32	–	0.15	–

The comparison of results shows, that present study gives the critical values of this design factor, bearing capacity factors, compared to, previous one researchers who did the similar analysis; like Budhu and Alkarni 1993 published in the Journal Geo-Technique, but our values are critical, because we considered one sided failure mechanism. Also the partial mobilization of passive earth pressure on the other side, as there is no failure surface is getting developed, at one instant of direction of acceleration of seismic horizontal acceleration.

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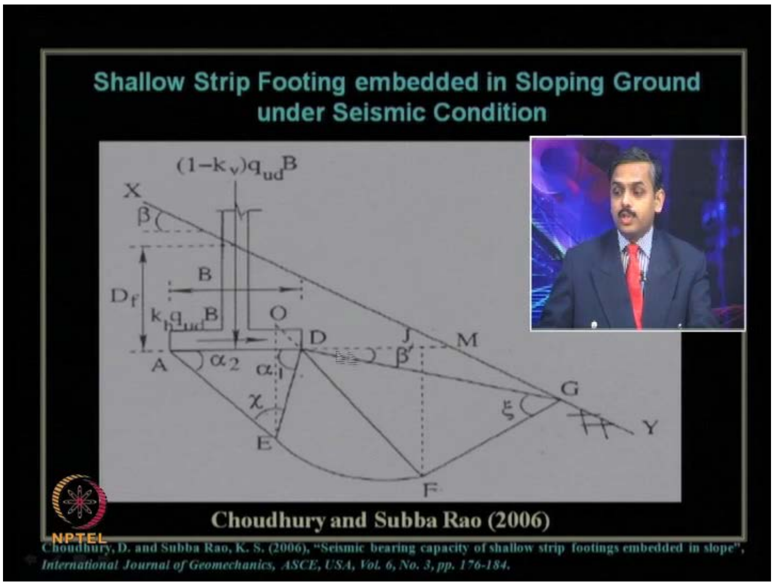

Then we extended our study for the shallow strip footing embedded, in the sloping ground. These are very useful in the hilly terrain; like in hilly region, like in Himalayan region, there are several houses which are constructed, in the hilly terrain or hilly region which are of sloping ground like this. So, how to make those foundations, seismically stable or earthquake resistant. The design methodology has been proposed, in this Journal paper Choudhury and Subba Rao 2006 which is published in International Journal of Geo-Mechanics, published by ASCE USA, this is the volume number and page number. Actually, this Journal paper of ASCE owns the best paper award from ISCMAG in 2008. So, this paper considered again one sided failure mechanism. And what will be further difference than the horizontal ground surface that, here may not be the full failure surface getting formed that is all zone 1, 2 and 3 are getting formed, because of limitation of this portion of the zone, which is available on the sloping ground.

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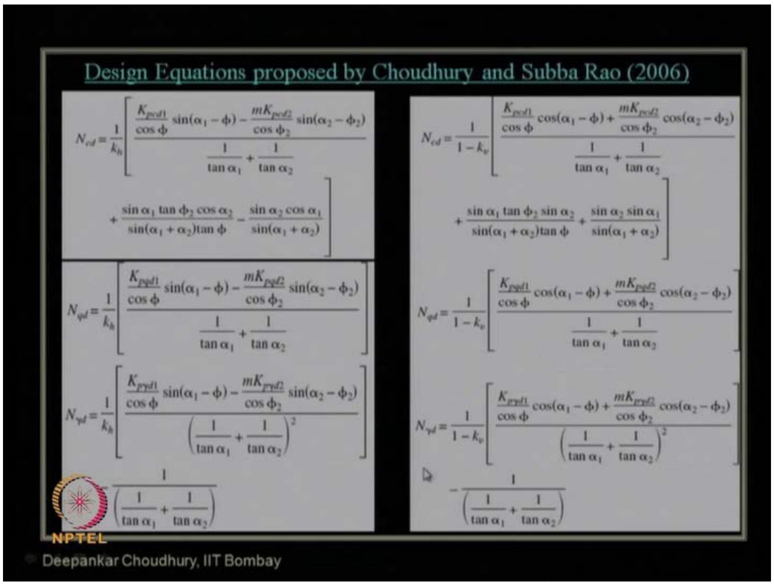

Based on that the equations are developed using limit equilibrium approach, considering both horizontal equilibrium and vertical equilibrium of all the forces involved, to find out the baring capacity factors N c d, N q d and N gamma d, considering horizontal as well as vertical equilibrium. Then what need to be done, in the analysis this parameters m and alpha 2, needs to be varied and based on those input parameters, the iteration techniques need to be adopted, for till this values of N c d computed by both the equations matches exactly same.

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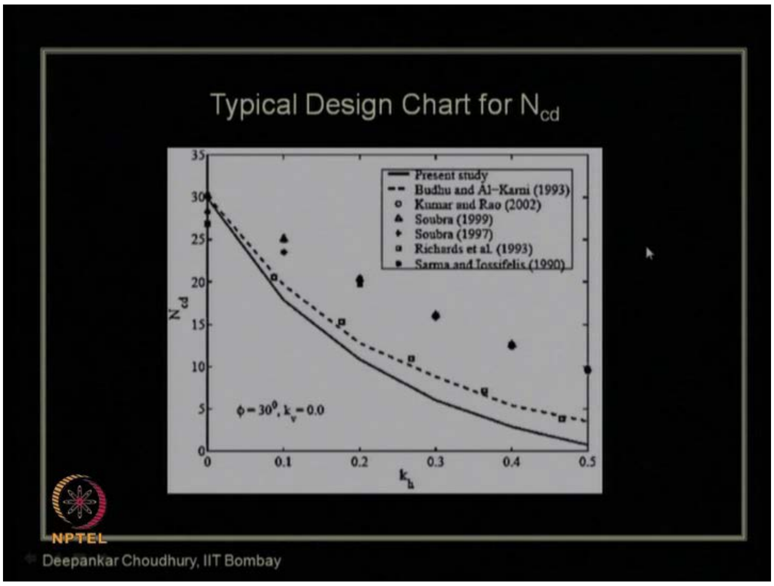

Similarly, for other baring capacity factors, and finally the design charts for this N c d, N q d and N gamma d are obtained, and as you can see, compared to other researchers results, as shown over here. The present study gives the minimum value of this N c d, that automatically shows that, our present study gives the critical design value, needs to be used at practice, for design of this shallow strip footing. Similarly, for N q d and N gamma d.

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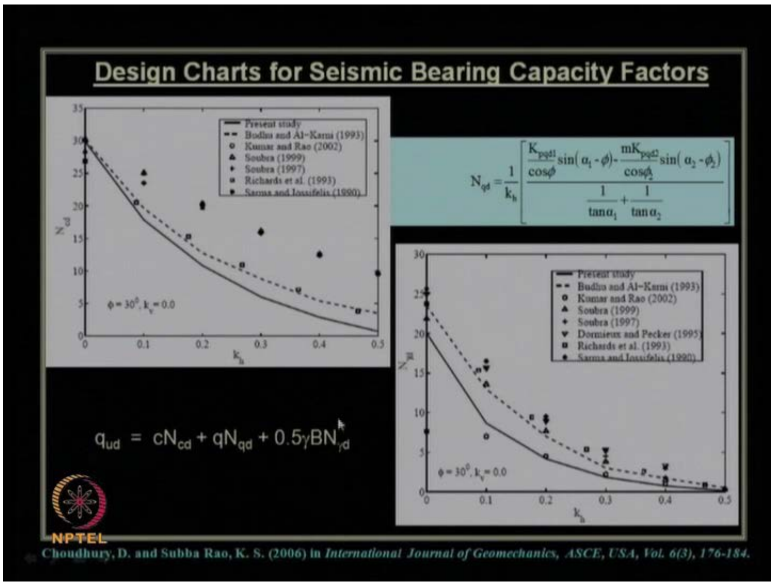

Also, this chart shows that this equation can be adopted, for the shallow strip footing in sloping ground condition.

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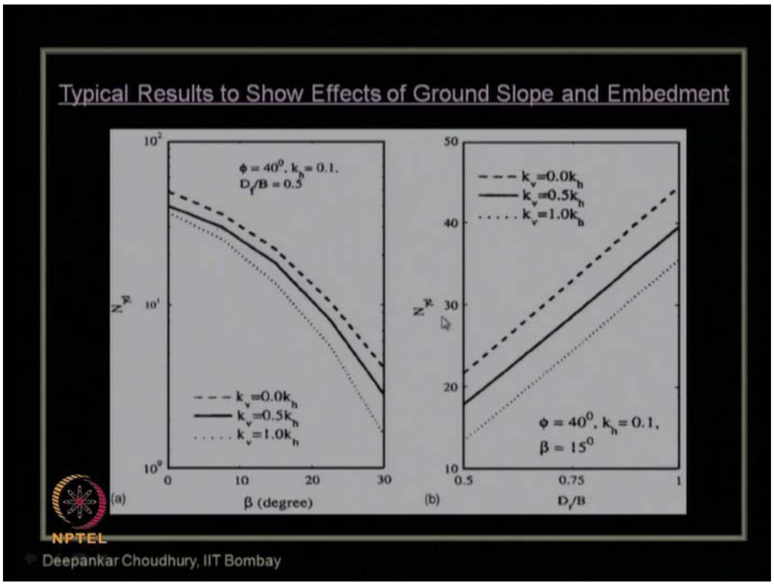

Also now this chart typically shows the effect of ground slope and embedment, which is necessary. Suppose this is the ground slope of beta, an increase of this ground slope, we can see the seismic bearing capacity factor N gamma d, decreases significantly, and so for the other bearing capacity factors, also for this chosen input parameters and different values of K v, what does it mean? As we know, as the slopping ground inclination increases, obviously the structure will be more vulnerable for instability; that is why N game d decreasing significantly, this condition needs to be considered.

Suppose at a place at a hilly terrain we have a sloping ground of 20 degree, then we should use the N gamma d value of this not the N gamma d of this, which is for beta equals to 0 degree at horizontal ground. Similarly the effect of the embedment depth D f is shown over here between D f by b ratio from 0.5 to 1, because it is a shallow footing. Within that range as it is expected, embedment depth increases means, the N gamma d value or the bearing capacity value increases, because more embedment is more stability. So, that is why, you can see over here, how much improvement of this bearing capacity can be obtained, from this results; that is, as we increase the embedment depth of this shallow footing, we will get, to use the higher value of seismic bearing capacity factor.

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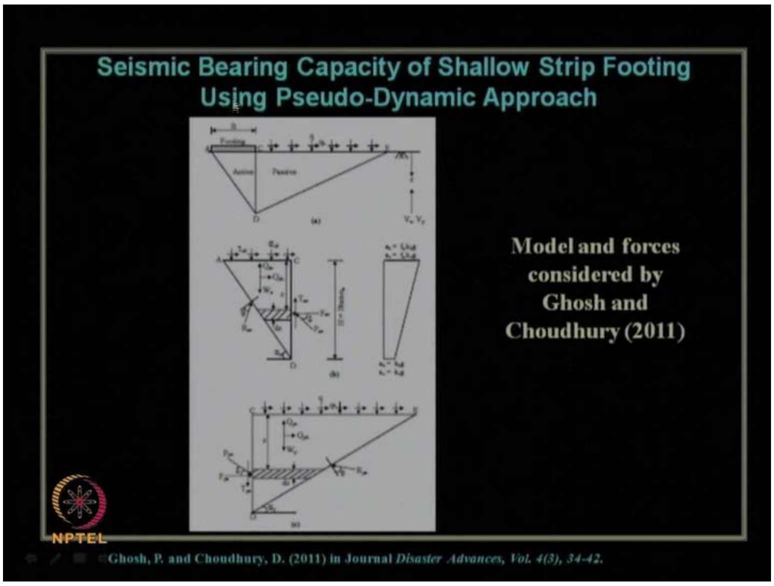

Now, let us look at the seismic bearing capacity of shallow strip footing, using pseudo-dynamic approach. Like, so far we have discussed about, the determination of seismic bearing capacity factors using, Pseudo-Static Approach. Now we are proceeding to pseudo-dynamic approach. This model and the forces considered, for determination of the seismic bearing capacity factors, for a shallow strip footing, in a cohesion less soil, was proposed by Gosh and Choudhury. This is a Prof. Priyanka Gosh from IIT Kanpur, who worked along with me, we are the collaborators. And our work has been published in this Journal; you can get details here, P Gosh and D Choudhury 2011 in the Journal Disaster Advances. This is the volume number and page number. So, what we have done, this is a shallow strip footing, the base of the footing is shown over here. The failure surface is assumed as a two zone failure surface, and again one sided failure mechanism, in the same way what we have considered in the, pseudo-static case, but instead of considering a curved failure surface, here we have taken the two zone; like active and passive.

This is basically was the extension of the pseudo-static bearing capacity factor, determination by Richards et al in 1993. Their paper we have extended for further analyzing it, through the use of this new pseudo-dynamic approach. So, within these two zones, these are the forces acting. here again within this active zone, we have considered this infinitesimal small horizontal slice, and finally, integrating over the entire depth of this active zone, we got this seismic inertia forces, considering the soil amplification also in this zone. As well has in the seismic passive zone, or this region, we obtain the corresponding inertia forces, in this zone considering the infinitesimal small horizontal slice, and then integrating over the entire depth of this passive zone. Then passive pressure has been estimated, which has been transferred to this place over here. So, considering this, and now active zone, this has to balance each other. So, that is the condition, at the interface the values should match. And then you will get what is the total capacity in terms of that q u d, what we have mentioned in the pseudo static case also.

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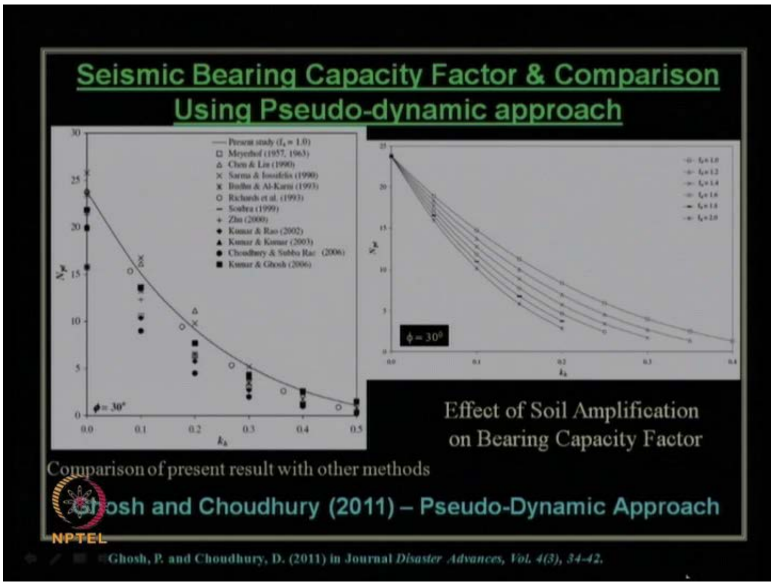

So, finally, what we have recommended, you can see over here now. Like seismic bearing capacity factor, and the comparison with other researchers results, those actually who have used pseudo static approach, because before this paper, nobody has used pseudo-dynamic approach for determination of seismic bearing capacity factor. So, this is the result you can see, this form line is showing the present study result; that is the results of Gosh and Choudhury 2011 is in pseudo-dynamic approach, for the bearing capacity factor N gamma d, as it varies with respect to K h values. Whereas, other different points which are showing over here, all other researches who have done the work; that is determined this N gamma d factor, using pseudo static approach. So, it is not necessarily that pseudo-dynamic will give us always the lower minimum. It depends on the combination as I said earlier.

So, we can see here pretty well that, even compared to the Richard's et al 1993, which method I have mentioned that we have extended from pseudo static to pseudo-dynamic. So these open circles, this points open circle. This gives little lower values compared to our pseudo-dynamic results. However, in pseudo-dynamic as we have mentioned, we can consider various dynamic related properties like; soil amplification, shear wave primary wave velocity, then duration of earthquake motion, frequency of earthquake motion and so on. Whereas the validation was, at K h equals to 0 and K v equals to 0; that is at static case, it value much match with that of Richard's et al 1993; that is a validation. So, that has been matched over here. But remember this present study gives us the result for amplification factor of one, so when there is no soil amplification.

So, in pseudo static method, we do not have the chance to incorporate that amplification factor, but we can take that aspect in the pseudo-dynamic approach. So, if you see the results of N gamma d, bearing capacity factor how it varies with respect to the variation of K h values, for different values of amplification factor staring from 1 to 2. You can see this, upper most value or upper most curve, is for no amplification, when there is no amplification that is amplification factor equals to 1. And the lower most curve is showing the value of N gamma d, when there is an amplification of two, what does it mean. Your design value of this baring capacity factor is, drastically reducing or significantly reducing, when there is an increase in the amplification factor in the soil. So, this aspect, we cannot address or we cannot incorporate in our pseudo static analysis.

So, suppose at a region, if from the euro code suppose you are using, the known value of the amplification factor, based on the site condition, say it is 1.4, and K h value say it is given as 0.2, or you have determined it from your seismic hazard analysis etcetera. Then you need to go to this curve, where it is this cross sign right; that denotes to f a value equals 1.4, you can see over here. So, this will be the value of your N gamma d, not this value. So, there is a huge change between these two values, you can see over here. So that means, the design will be more critical, in terms of, when we are considering the soil amplification, which needs to be incorporated, and that can be incorporated only by using this pseudo-dynamic approach. Now, let us move to our another subtopic; that is seismic stability of finite soil slopes; like we will be talking about the stability aspects of soil slopes.

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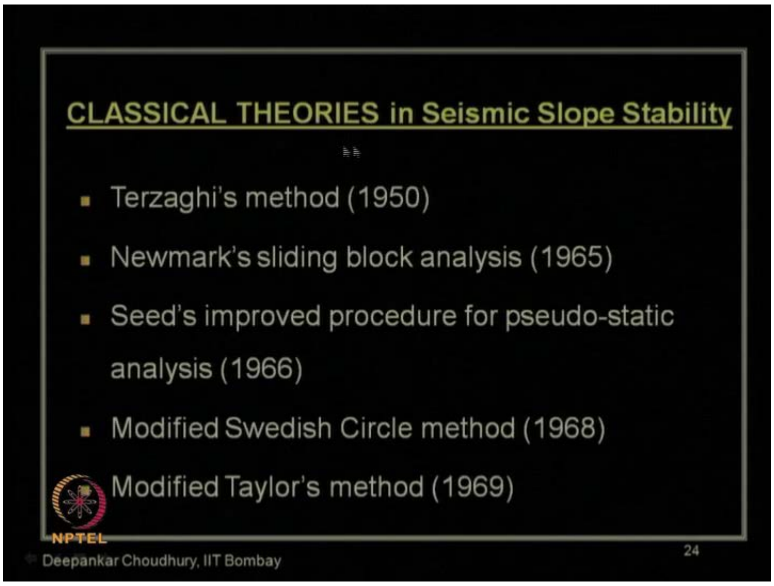

Let us see the classical theories in seismic slope stability analysis. There are several slope stability theories as we all know, but we will only considered those, which deals with the seismic theory; that is the extension of the static slope stability analysis to the seismic level. All are by considering of course, Pseudo-Static Approach or Quasi Static Approach. The first one is given by Terzaghi in 1950; then immediate next one, which is still widely used around the world, is Newmark's sliding block analysis, which was proposed in 1965. I will go through this method very thoroughly, because this is one of the basic research on this seismic slope stability problem, and that is why even today also because of its simplicity, and the possibility of determination of displacement as well, this method is widely used around the world. Then seed's improved procedural for pseudo-static analysis, then modified Swedish circle method and modified Taylor's method. There are several other methods after that which have been developed, I will try to discuss, most of the recent approaches for this seismic slope stability analysis.

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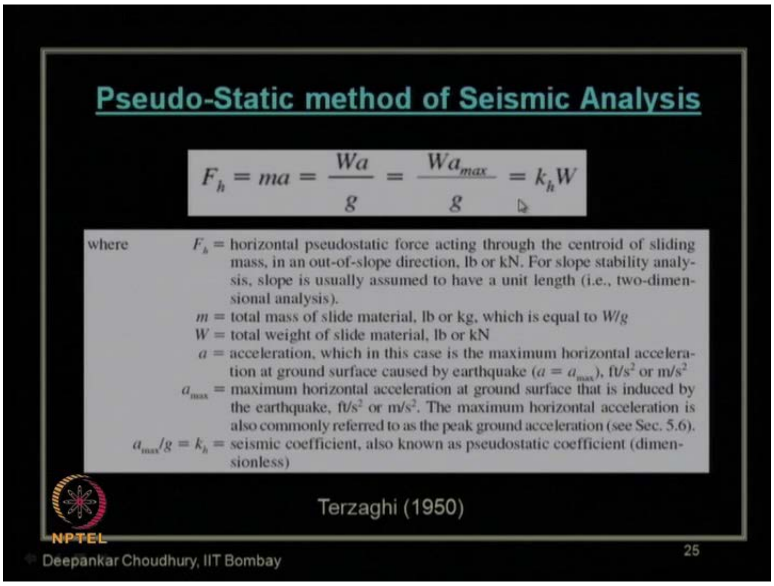

Coming to the Terzaghi's concept of pseudo- static method, which I have already discussed, in our present module in one of the previous lecture; as we all know pseudo-static method of seismic analysis is derived, something like that. This is the wave we calculate the horizontal inertia force, seismic inertia force F h is nothing but mass times seismic acceleration in the horizontal direction, which you can express as W times a by g, W is the weight of the failure zone, on which it is acting, the seismic acceleration is acting. So, a generally we take the maximum acceleration, a max by g. So, this a max by g ratio is called K h; that we have already seen, this is called coefficient of seismic horizontal acceleration.

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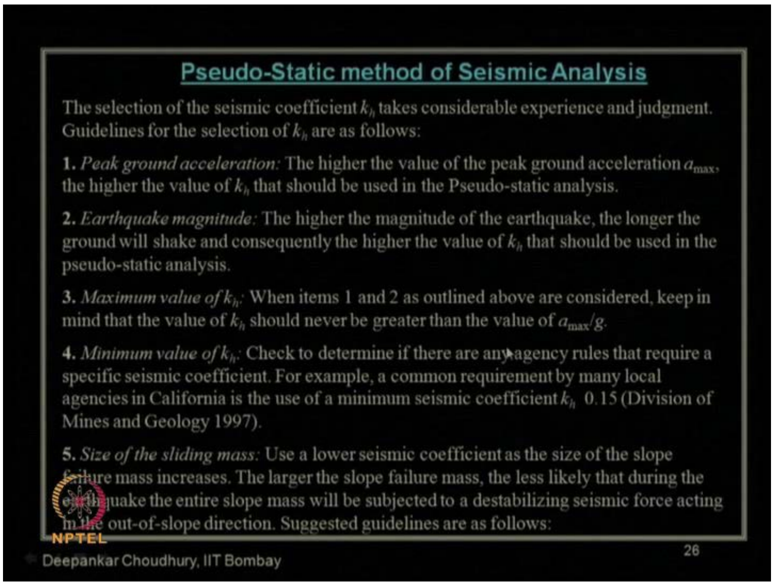

Now how to select this K h value, already I have discussed this though, but here some other points and recommendations, as given by various researchers. Let us look at here various guidelines for the selection of K h value for any pseudo-static analysis. Like it is, will be based on peak ground accelerations some people say. The higher the value of the PGA that is a max the higher will be, the value of K h, and that should be used in the pseudo-static analysis, but remember using the exact value of a max as K h will be a gross approximation, or it is it will give you an uneconomic design, because that is a two higher value which is not sustaining for a longer duration. So, we need to evolve certain criteria based on which we can select this K h value.

As I have already mentioned the euro code criteria, how to estimate this K h value, based on different factors etcetera. There are other codes also worldwide, which suggest how to find out this K h value. Now based on the earthquake magnitude, the higher the magnitude of the earthquake, the longer the ground will shake and consequently the higher the value of K h; that should be used in the pseudo-static analysis. Now, maximum value of K h, when item number this 1 and 2, has outlined above or considered and kept in mind that the value of K h, should never be greater than this max by g; that is quite obvious, because this is the absolute maximum value possible for K h to have. Now, minimum value of K h, what should be the minimum or threshold value?

It needs to be checked, if there are any agency rules that requires specific seismic coefficient, like as we have mentioned there can be some guideline for threshold value of acceleration; that will be your minimum value. Like local agencies of California uses the minimum seismic coefficient K h of 0.15 for the division of mines and geology as for as 1997 recommendations are concerned. Size of the sliding mass, now use a lower seismic coefficient as the size of the slope of failure mass increases; that means, if you consider, the extension of failure zone for a larger area, you can go for a lower seismic coefficient. The larger the slope failure mass, the less likely that during the earthquake, the entire slope mass will be subjected to the destabilizing seismic force, in the out of slope direction.

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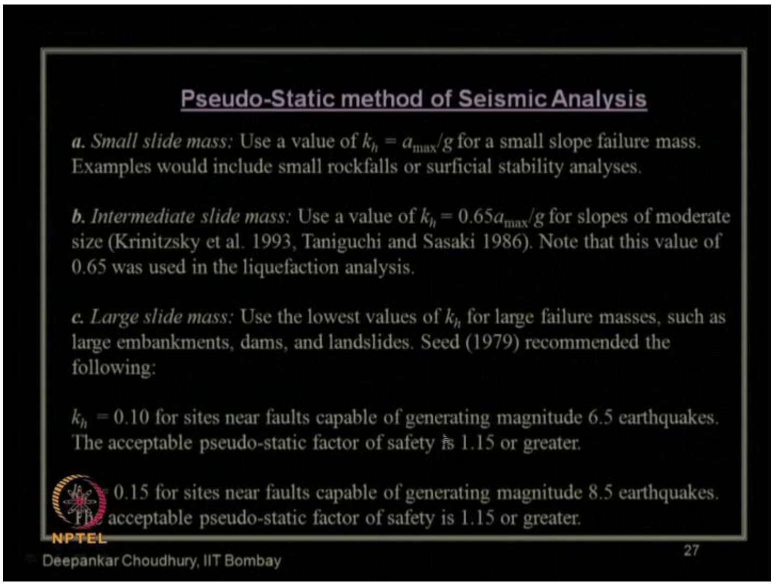

So, what are the other suggestions; like when it is a small slide mass, then recommendation is use K h equals to a max by g, but when it is intermediate sliding mass, use the value of K h as 0.65 times a max by g; that means, 65 percent of this maximum value. And remember, this factor of 0.65 is also used in our liquefaction analysis which we have discussed earlier in, one of our module. And large slide mass, here use the lowest value of K h for large failure masses; such as like large embankments, dams, land slide etcetera. And seed in 1979 recommended that K h value should be 0.1 for the sites, where near falls capable of generating magnitude of 6.5 earthquakes is possible, and in that case he recommended, that the acceptable pseudo-static factor of safety should be 1.15 or greater.

That means, minimum value of factor of safety should be 1.15 for that slope, that is for design of large embankment or dam or land slide problems. Whereas, K h value needs to be used as 0.15, for the sites near falls capable of generating magnitude of 8.5 earthquake. So, for 8.5 earthquake, K h value of 0.15, for 6.5 earthquake K h value of 0.1. Remember these are recommendations for only large slide mass, where large mass is involved. So this is on the lower side of the ranges of K h, which can be used for design. So, these are the recommendations, which people can use for practical design purpose, unless they have a strict or very clear guideline about the section of the K h value, from the site response and from the local earthquake data.

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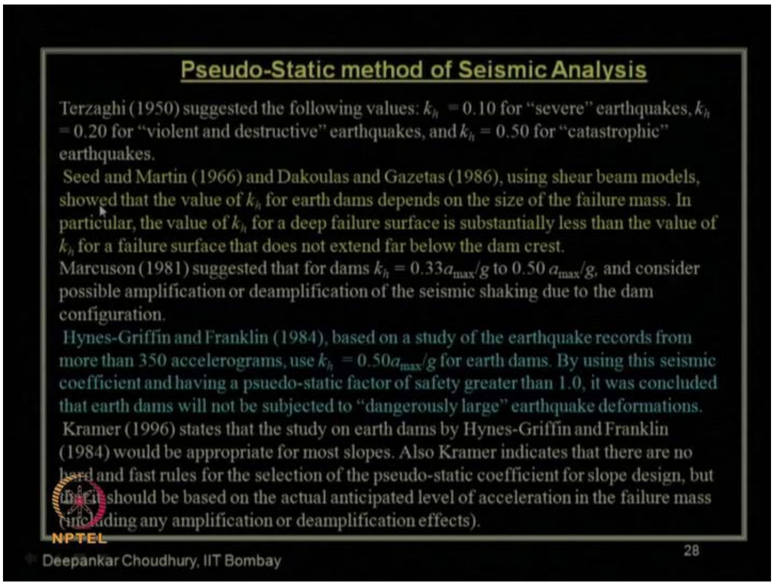

Now other recommendations like Terzaghi's 1950 suggested; that the K h value should be considered as 0.1, for severe earthquake. K h value should be considered as 0.2 for violent and destructive earthquake. Whereas K h value should be used 0.5, for catastrophic earthquake, so those are Terzaghi's recommendations. But remember these recommendations of pseudo-static values are way back in 1950. So, in today's design method, if somebody are using these values, remember it will be primitive in nature. So, you can use it as a first step of your design, but you have to rely on the local seismic motion, local seismic hazard analysis, local site response analysis, and all these studies, so that you get a proper input value of your K h for design, for this pseudo-static design.

Other researchers like, you can see over here Seed and Martin in 1966. Then Dakoulas and Gazetas in 1986, they used the Shear Beam Model and showed that the value of K h for earth dams depend on the size of the failure mass. And in particular the value of this K h for a deep failure surface is substantially less, than the value of K h for a failure surface, that does not extend far below the dam crest; that means, for shallow failure, you use the higher values of K h, for deep failure you use a lower value of K h; that is what it means. And Marcuson in 1981 suggested that for dams, you use the K h value of between 0.33 times of that a max by g to 0.5 times of a max by g.

Remember these are the values corresponding to with, some factors as proposed in the present day euro code. Can you see these values, this coefficient 33 percent and 50 percent of that maximum value, with some influence factor etcetera, which we have discussed earlier. Now, other researchers they mention that, based on their recorded 350 accelerogram data in 1984. This Hynesand Griffin and Franklin in 1984 proposed the k h value should be used as, 0.5 times of a max by g for design of this earth dams. But by using this seismic coefficient, and having a pseudo-static factor of safety greater than one. It was concluded that earth dams will not be subjected to dangerously large earthquake deformation, based on their analysis, based on only this 350 limited accelerogram data, remember that.

Whereas, Kramer in 1996 states; that the study on the earth dam by this researchers, would be appropriate for most the slopes, because slope stability problem, is also applied for earth dam also, for the stability of the earth dam slope. So, also Kramer indicates, that there are no hard and fast rules for the selection of this pseudo-static coefficient for slope design, but that it should be based on the actual anticipated level of acceleration, in the failure mask, including any amplification or de-amplification effect. So, remember this guideline or suggestion, as given in Kramer in 1996, that when you have some actual anticipated level, based on your local site response analysis and seismic hazard study, you should use that, where you can include this amplification and de-amplification effect also.

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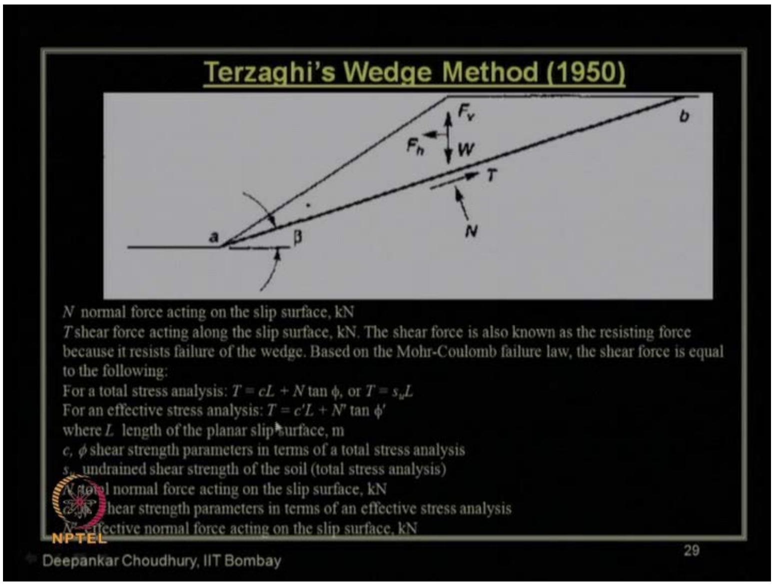

Now let us go through Terzaghi's wedge method for the slope stability problem, which was proposed in way back 1950. This is the basic slope, you can see over here. This is the slope, soil slope, and this, the failure plane, assumed failure plane by Terzaghi's was considered as a triangular wedge. So, this is the failure mass of the wedge, weight of this failure zone is W, and the pseudo-static seismic inertia forces in horizontal direction is F h, and in vertical direction is F v.

And remember it needs to be considered in both the direction; like this way, as well as this way, and F v also need to be considered this upward as well as downward. And for a particular combination of these directions of this F h and F v, you will get the critical value of factor of safety, or lowest value of the factor of safety for your design. So, what are the other forces over here, this N is nothing but the normal force, which is acting on this planer failure surface, and T is the shearing force acting at this portion. So, from the total stress analysis, one can easily write that T is nothing but C times L plus N time phi. Where C is the unit cohesion of this soil, and L is nothing but length of this failure plane; that is a b, length of this a b, and phi is the friction angle of this soil. And for effective stress analysis, the values will change to C dash L and N tan phi dash.

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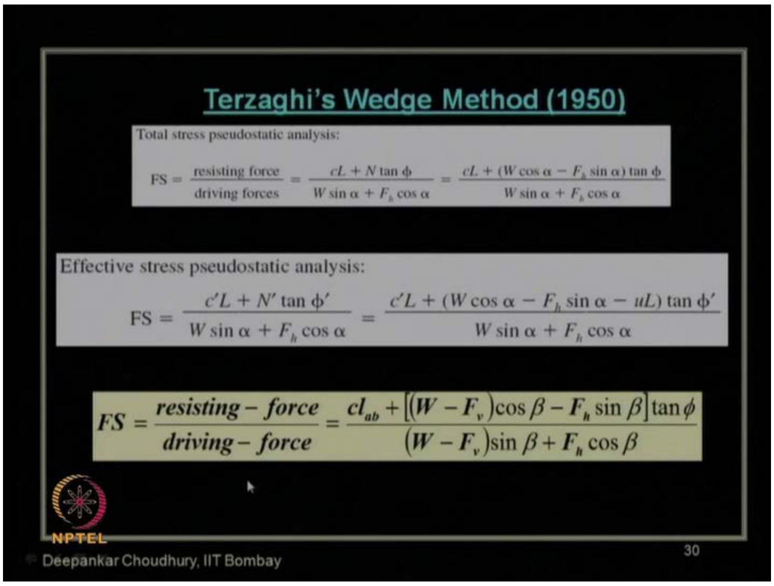

Now, what is a definition of factor of safety as we all know. The definition of factor of safety is nothing but ratio of resisting force to driving force. Now what is resisting force over here, and what are the driving forces. Let us go back, this seismic force when it is acting in this direction, it is trying to displace the slope, it is trying to fail the slope. So, this is the driving force. Whereas, this shear strength; that is the property of the soil which it is coming, or which it is providing the support, that gives the resisting force. So, from the resisting force criteria using total stress analysis, it will be c L plus N tan phi. And what are the driving forces, it will be in that direction, it will be F h cosine of alpha plus W sine alpha. Look at here, this component of W and F h component in this direction; that will try to fail it in this direction.

So, if we simply this further what will be the value of N. N is nothing but the normal force how you will get. Normal force will be this W is acting cosine component of that, and this F v has to be deducted, if this is the critical direction. So, F h a W cos alpha minus F h sine alpha of tan phi, this is without considering F v. Now, if you have F v in driving force also you will get another component of F b, in resisting force also you will get another component of F v, this is without considering the vertical one. And with consideration of the vertical one, you can see the factor if safety is given over here. The resisting force by driving force c times L length of that a b, failure zone plus W minus F v times cosine of beta minus F h sine beta, of tan phi, because this is nothing but your N.

Look at this, these are simple mechanics, you can see resolve the forces over here, and you can get what is the value of N. N is nothing but equals to, this W cosine alpha, and in this case if you denote this angle as beta, then W cosine of beta, minus of F v cosine of beta, because F v can be plus and minus, you have to consider the critical value, for which you are getting the minimum factor of safety. And F h sine beta; that is also in the other direction, it is giving a component of N. And what are the driving forces, driving forces from F v as I said there will be a component. W minus F v sine beta will be one component, and F h of cos beta will be another component. So, using this formula, one can easily calculate the factor of safety, and what is the recommendation as we have seen. For safety it has to be 1.15, more than that is safe factor of safety against pseudo-static seismic analysis.

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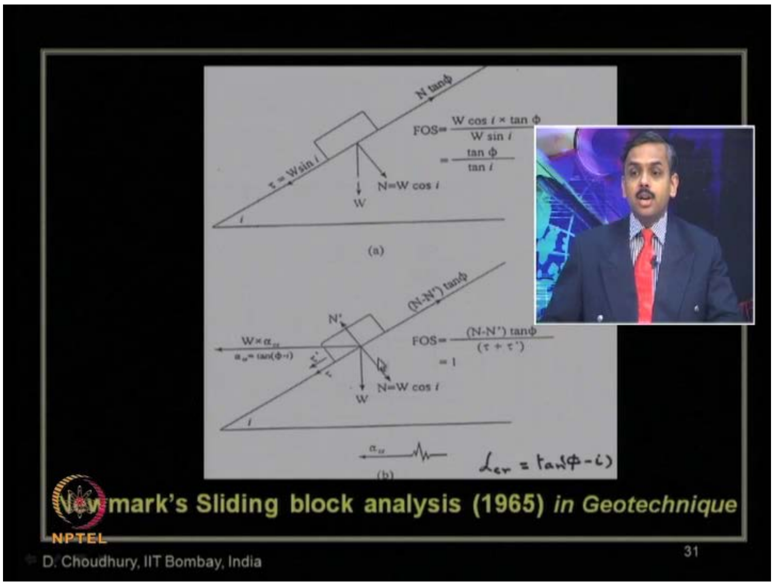

Now, let us come back to another methodology, which was proposed by Newmark. So, this Newmark's sliding block method, is one of the pioneering work in the area of this slope stability analysis for soil, how it was developed let us see. It was developed based on the concept of friction block; that is a block which tends to move over a sloping ground, like this, what are forces acting on it, based on that mechanics, Newmark had developed this problem. So, what was considered this mass, is the failure mass or failing mass, which is failing with respect to a stable zone of soil; that means, if you consider this portion is a stable zone, this mass is failing with respect to this plane.

Now, what is Newmark's sliding block method, why the name sliding block. As the name suggest this is the block, soil block; that is the failure mass, which is sliding or moving over this stable bloke. So, it was developed in the year 1965, in the Journal paper, appeared in the Journal geo-technique, published by IC London. This is a classical work as I already mentioned. You can see the weight of the failure mass is W, so if you resolve it into two directional components, N; that is your normal force is nothing but W times cosine of i, i is this inclination, and this force is W times sin i. This W sine i is trying the block to slide down, and what resist it. So, that is the driving force, and what resist it, that is the shear strength property.

What is the shear strength property, what is the fictional property; that is nothing but this normal force times, tan of phi, tan of phi because it is the same soil, soil over soil. Otherwise it would have been delta F. Suppose, there are two materials; like we solved in our basic engineering mechanics problem, friction problem; that a block is sliding over another material, there will be interface friction between the two materials. So, in case of our soil it is N time phi; that is the stabilizing force. So, the factor of safety is expressed as stabilizing force by driving force. So, stabilizing force in this case is, N times tan phi, and N is W of cosine of phi, divided by driving forces W sine i. So, if you simplify it, the factor of safety is simply just tan phi by tan i, remember this is for the static case. So, initially Newmark has shown this figure, to make everybody understand, that this is a simple extension of this static problem of friction sliding block method, to the seismic analysis for the soil slope.

Now additional factor what are coming into picture, this W times alpha c r, what is that. This is nothing but inertia force seismic inertia force in the horizontal direction. Newmark has considered only the horizontal seismic inertia force. He did not consider the vertical one, but later on people had modified this Newmark sliding block method, considering vertical force also, that you people also can do, because it is nothing but a simple mechanics problem. So, when this seismic horizontal inertia forces added, what are the changes occurring. Now, you have additional component of this W times alpha c r, which is the inertia force, along this driving direction. And you have another component of this, in the vertical direction, which is reducing your normal force of this N.

So, both ways it is, damaging the stability, can you see that, because this is adding to this denominator on this driving force this dash, and it is reducing the stabilizing force in terms of this N dash. So, using this factor of safety can be calculated. Now Newmark has gone further, beyond what Tarzagi has recommended. Terzaghi's stopped here, that is he mentioned determined the factor of safety, and check the factor of safety if it is more than 1.15, then it is a stable slope, but if it is not stable, then how much is the displacement. Up to how much we can consider that it is allowed to fail; that is the amount of displacement cannot be considered in Terzaghi's method, but Newmark's sliding block method, the beauty is, it is a displacement based approach also, including the force based concept.

Newmark proposed that equate this expression of this factor of safety equals to one. And when you are equating with respect to one, whatever value of this alpha c r; that is this acceleration coefficient, seismic acceleration coefficient we are getting for a value of factor of safety equals to 1; that is called critical acceleration or yield acceleration. For that yield acceleration, beyond that acceleration, if actual acceleration is beyond that value, then the factor of safety will be of course, less than 1; that means, it is no longer a stable slope. It will start sliding down, then how much is that sliding, how much is the displacement that can be computed, based on the difference of the acceleration between this critical value, and the actual value of acceleration. Now integrating that acceleration twice you will get the displacement.

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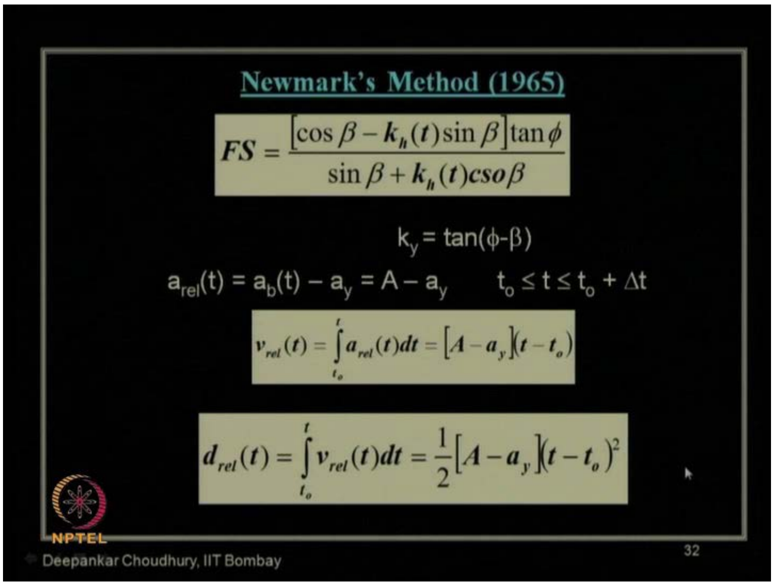

So, what I am telling, this is the expression of factor of safety as per Newmark's method. Newmark introduce this coefficient K y, which is called yield acceleration coefficient. This yield acceleration coefficient is nothing but that seismic acceleration coefficient for which factor of safety equals to 1. And when your actual acceleration is say capital A acting on the slope, you have to find out, what is difference between this A minus a y, a y is nothing but K y times g. So, this is your relative acceleration, which is making the slop to slide down or to fail. So, that relative acceleration, which varies between times scale of t naught to t naught plus delta t. You can integrate it between the time scale to get the relative velocity.

Further, you can integrate that velocity between that time scale, to get the relative displacement. So, this value of relative displacement will give you the, movement of the failed slope which is having, factor of safety less than 1. So, this is also very important nowadays, as for as performance based design or the displacement based approach of design is concerned. So, with this, we have come to the end of today's lecture, we will continue further in our next lecture.