Geotechnical Earthquake Engineering

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Module - 9

Lecture - 36

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. We are currently going through our module number nine, which is seismic analysis and design of various geotechnical structures. Within that module, we were discussing seismic design of retaining wall.

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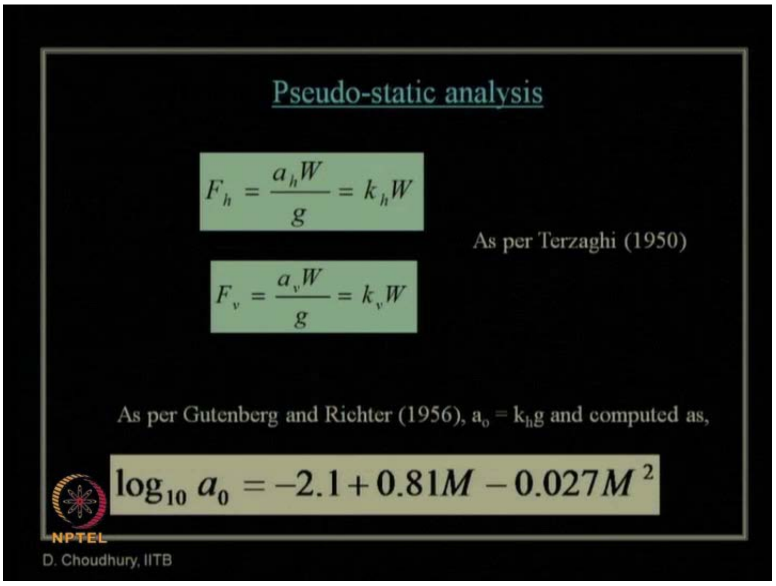

So, a quick recap what we have learnt in our previous lecture. We have mentioned about mostly force based analysis and displacement based analysis. And within force based analysis we mentioned pseudo static analysis is the basic method by which the analysis for the seismic earth pressure acting on the retaining wall is computed.

So, that pseudo static analysis was originally proposed by Terzaghi in 1950, where the seismic inertia force in horizontal and vertical directions are calculated by using these expressions; where a h is seismic horizontal acceleration, W is the weight of the failure mass, by g is the acceleration due to gravity. So, a h by g can be considered as k h, which is called coefficient of horizontal seismic acceleration times W. And the seismic vertical inertia force is calculated like a v times W by g; where a v is vertical seismic acceleration, g is acceleration due to gravity. So, a v by g can be considered as k v. k v is called coefficient of vertical seismic acceleration times, the W is mass of the or weight of the failure zone.

So, that way we compute the seismic horizontal and vertical inertia forces. And Gutenberg and Richter in 1956 gave a generalized expression how to estimate the value of this a. They have given this expression, where this log to the base ten a naught is given by this expression; where M is the magnitude of earthquake in the closed vicinity of an epicenter. Remember, this equation is only valid in the closed vicinity of the epicenter region and for a shallow earthquake and M is the local magnitude or Richter magnitude scale of the earthquake.

So, if somebody knows the local magnitude of earthquake and if it is closed to epicenter they can compute the value of a naught. a naught is nothing but they mentioned it is nothing but horizontal seismic acceleration which can be expressed as k h by g as I have already mentioned. So, this is one way of calculating k h. Other way, as I have already mentioned it can be taken as proportion or a component or a fraction of the p g a value. That is, peek horizontal and peek vertical acceleration. But most of the times, this k v is taken as a function of k h; that is either 0 times k h or half of k h or two-third of k h or full one times of k h; that means, k v will be always less than or equals to k h.

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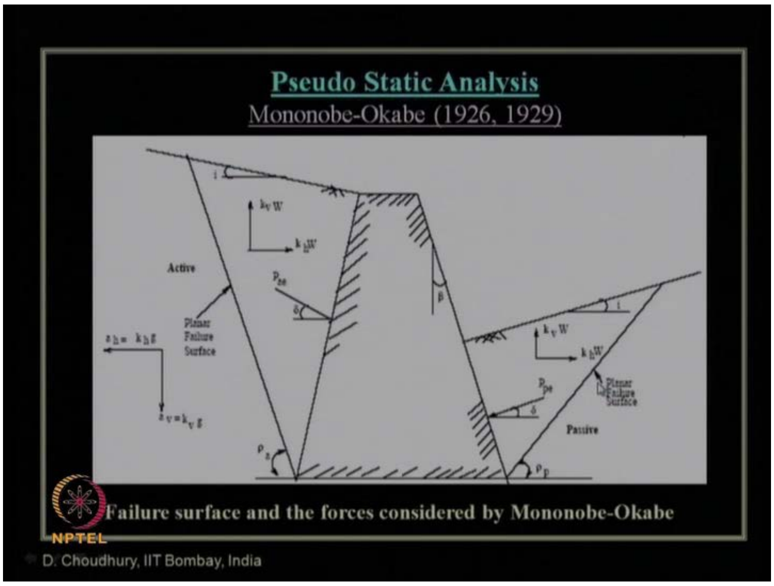

Then, we have studied in detail the basic pioneering work proposed by Mononobe and Okabe. Mononobe and Matsu proposed it in 1929 and Okabe proposed it in 1926; the pseudo static approach for the rigid retaining wall, gravity type retaining wall like this. This is the active state; this is the passive state. And for planer rupture surface with the cohesionless soil only, how to estimate by considering this pseudo static coefficients in the failure mass, how to estimate the seismic passive and active earth pressure.

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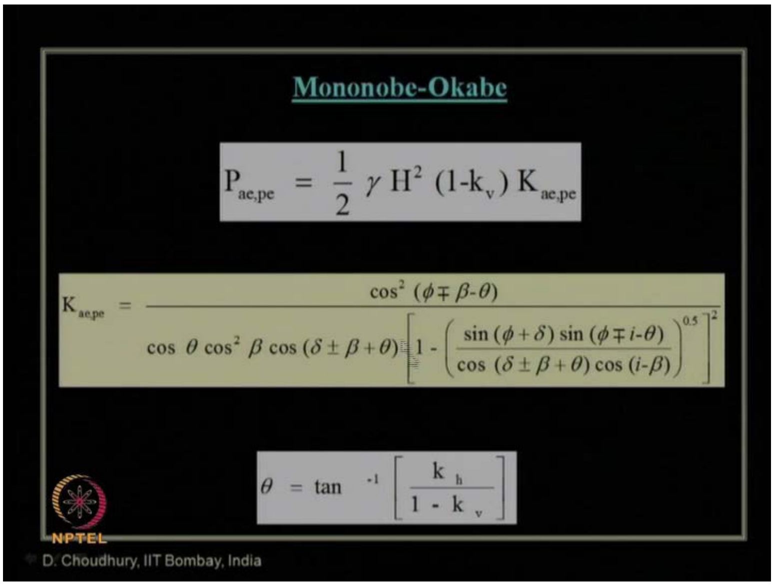

So, these are the final equations or expressions proposed by Mononobe-Okabe. This is the total passive and active earth pressure. So, P p e is nothing but seismic passive earth pressure and P a e is nothing but seismic active earth pressure. Total value of the pressure is half gamma H square times 1 minus k v times Ka e and or K p e. That is, for active case it is K a e; that is, coefficient of active earth pressure coefficient on the seismic condition. And K p e is seismic passive earth pressure coefficient, k v is the vertical seismic acceleration coefficient, H is the height of the wall, gamma is the unit weight of the backfill material.

So, the expressions of Ka e or K p e is given like this. So for a e, one should use the upper symbol; that is here the minus, here plus, here plus and here minus. Whereas for K p e, one should use the lower symbol; that is here plus, here minus, here minus and here plus; where in this expression, theta is nothing but tan inverse k h by 1 minus k v. And other parameters like, delta is wall friction angle and phi is the soil friction angle and this beta is nothing but the inclination of the wall, which is called wall batter and I is nothing but inclination of the ground.

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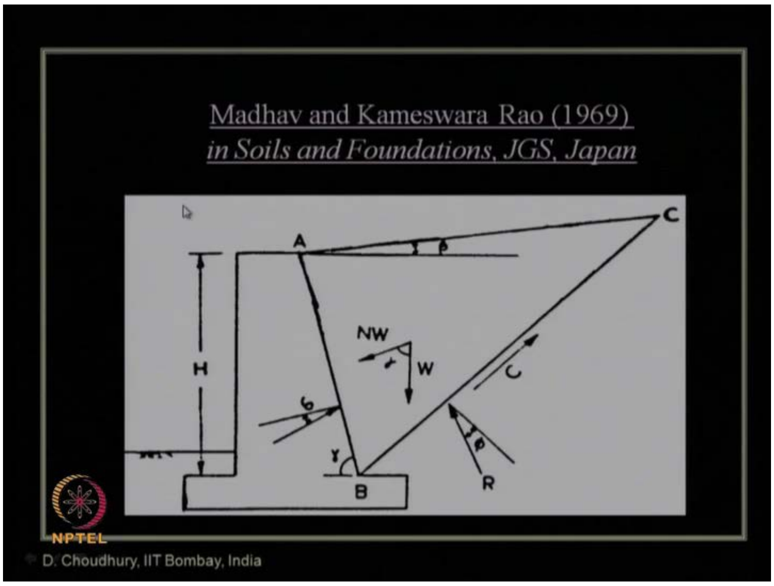

Other researchers like Madhav and Kameswara Rao in 1969 in their journal publication in oils and foundations published by Japanese Geotechnical Society in Japan. They also estimated the seismic active and passive earth pressure using this pseudo static approach.

But they have used C phi type of soil that is the generalized soil, whereas Mononobe Okabe approach was only for the cohesion less soil.

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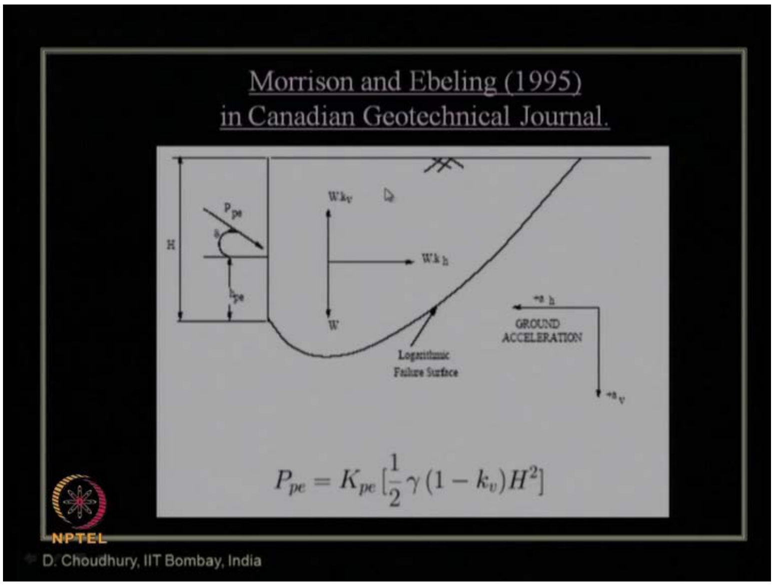

Then Morrison and Ebeling in 1995 in their journal paper "Canadian Geotechnical journal", they proposed for a rigid retaining wall how to estimate the passive earth pressure considering curved linear. That is instead of a planner failure surface, curvilinear failure surface and the shape of that curvilinear failure surface they considered a logarithmic failure surface and they estimated the pseudo static seismic passive earth pressure acting on the retaining wall using limit equilibrium approach.

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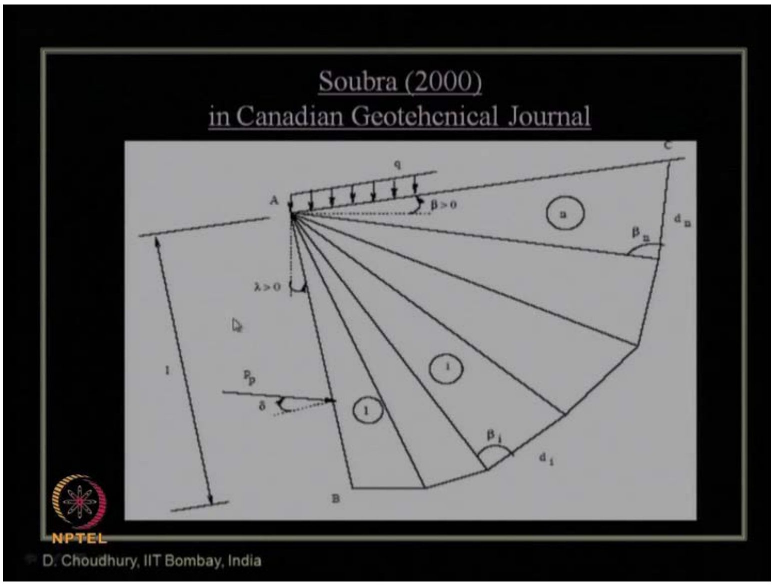

Then Soubrain 2000, which is also published in the journal “Canadian Geotechnical journal”, found out the seismic passive earth pressure coefficient acting on a rigid retaining wall by considering curvilinear failure surface. But this curvilinear surface was proposed to be a piece wise linear like wedges like this. These are piece wise linear as you can see. So, if you increase the number of these triangular wedges, obviously it will give you more curvilinear surface.

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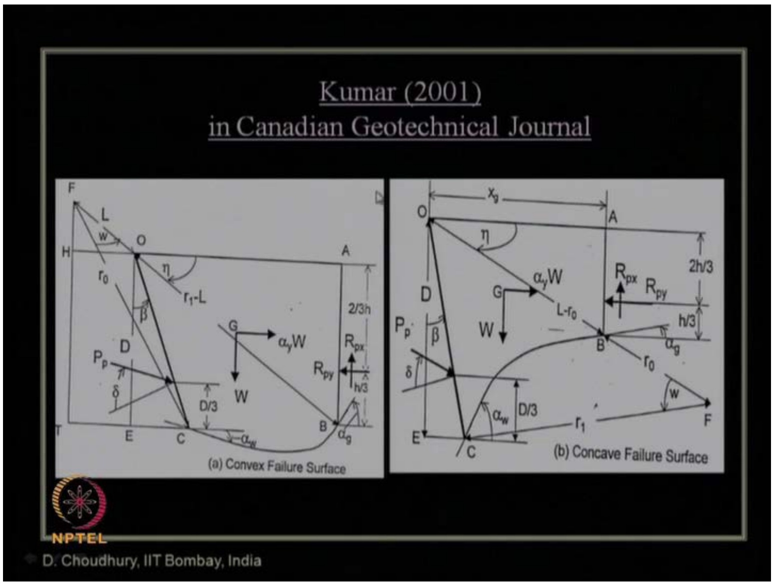

Then Kumar in 2001, which is also appeared in “Canadian Geotechnical journal”, estimated the seismic passive earth pressure coefficient based on two types of failure surface. One is convex failure surface; another is concave failure surface, which I have referred in the previous lecture as positive and negative wall friction angle cases.

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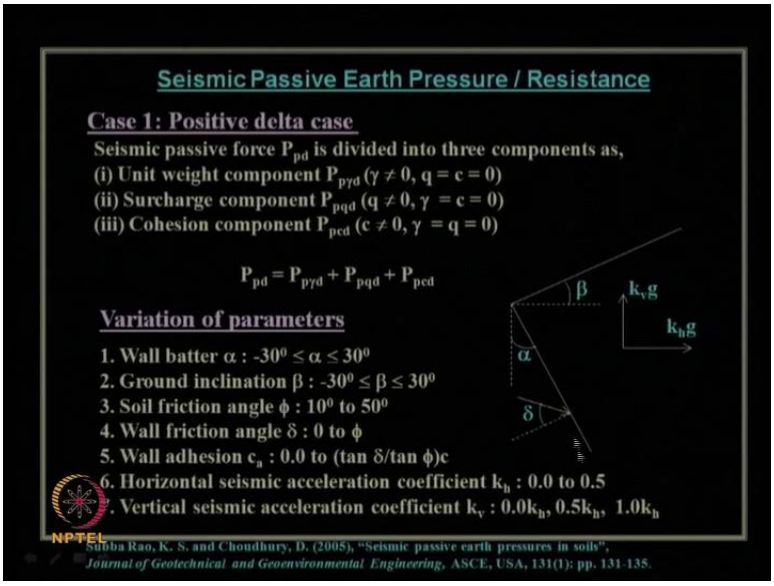

Then, our joint research work; that is, it is a part of my Ph.D. work with under the supervision of my supervisor K. S. Subba Rao at IISC Bangalore; this publication, Subba Rao and Choudhury 2005 in ASCE journal of Geotechnical and Geo environmental Engineering. We considered the estimation of seismic passive earth pressure for positive delta case. We have explained what is positive delta case the total seismic passive earth pressure is estimated using three components. One is unit weight component; another surcharge component and another is cohesion component; that is a generalized case has been considered for a C phi soil with the surcharge acting on the ground.

So, if any component is absent, one can easily take the corresponding value of the coefficients. And these are the ranges of the wall batter. This is the wall batter, wall inclination. This is the ranges of the ground inclination which are considered. So, these are parametric studies was carried out; soil friction angle range, wall friction angle range, wall adhesion. Wall adhesion is also a function of the soil cohesion with respect to the wall friction angle, with respect to these value and horizontal seismic acceleration coefficient with this range and vertical seismic acceleration coefficient in this range of parameter.

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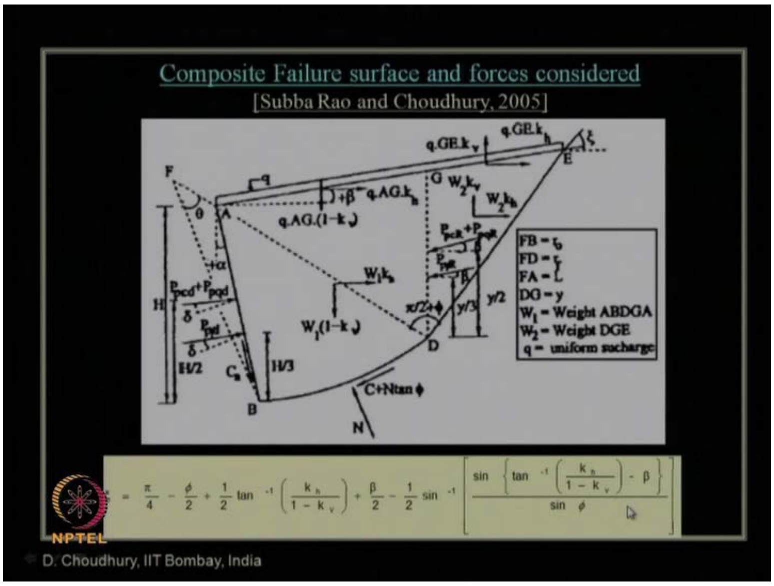

So, this was the model. As we have already seen in our previous lecture, that is the composite failures. Curved rupture surface was considered for the case of passive earth pressure. And the exit angle was derived as given by this expression.

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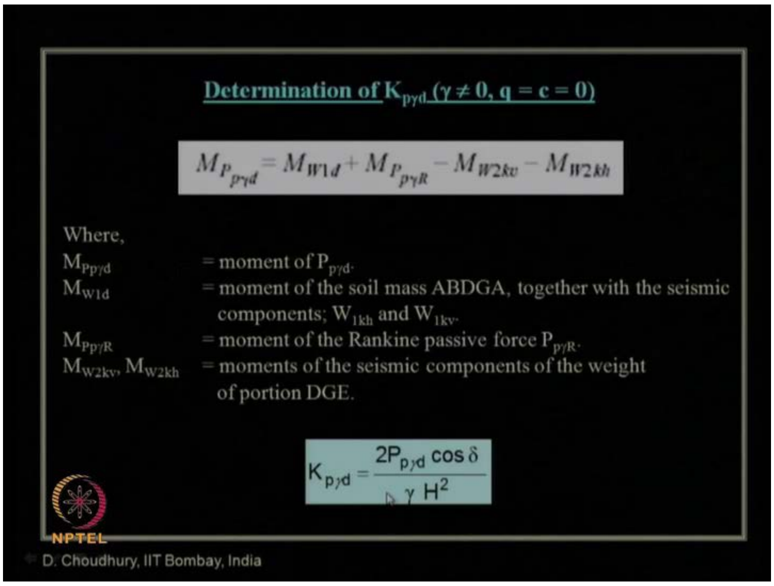

Then, the individual seismic passive earth pressure coefficients like K p gamma d in terms of unit weight, K p q d in terms of surcharge and K p c d in terms of cohesion component were estimated.

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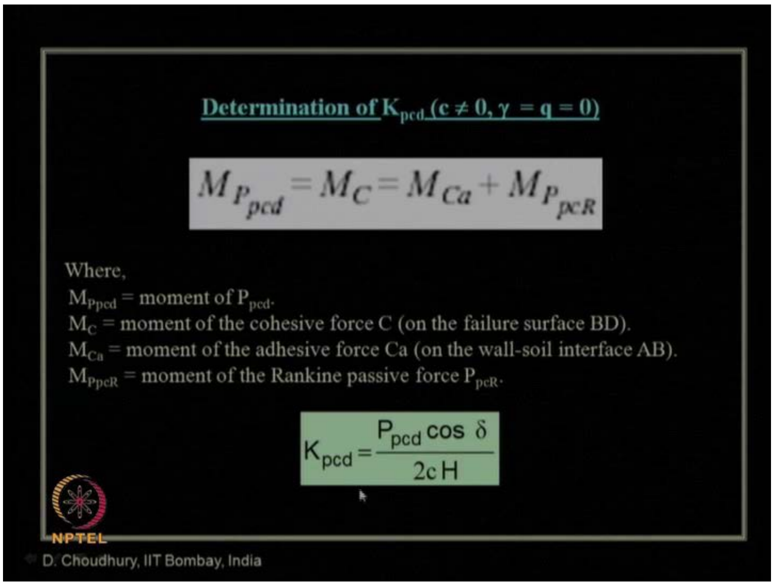

So, these are the design values of the seismic passive earth pressure coefficients.

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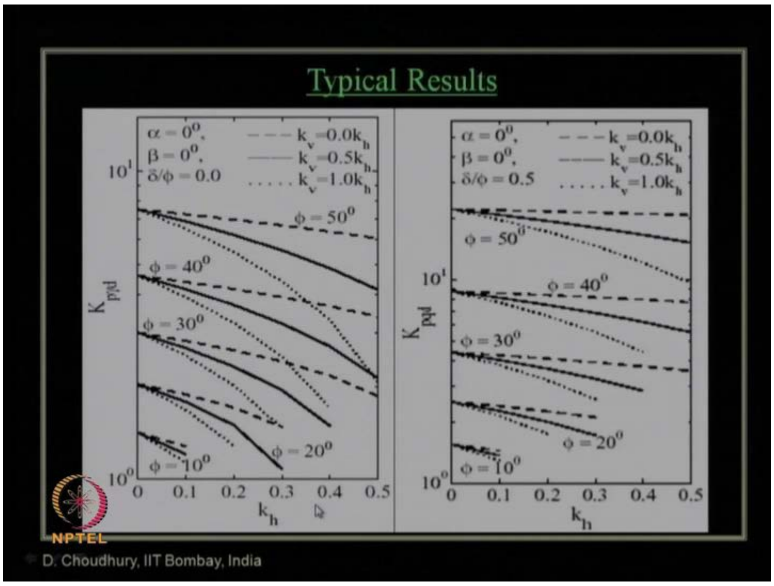

And these are the design charts, which any practitioners or designer can easily use. As I have said knowing the value of k h, input value as per the seismic zone is concerned and the design value of k v as I said… Suggests k v to be considered as two-third of k h typically, and knowing the soil, what is the value of the phi of soil, one can get the value of K p gamma d from this design chart.

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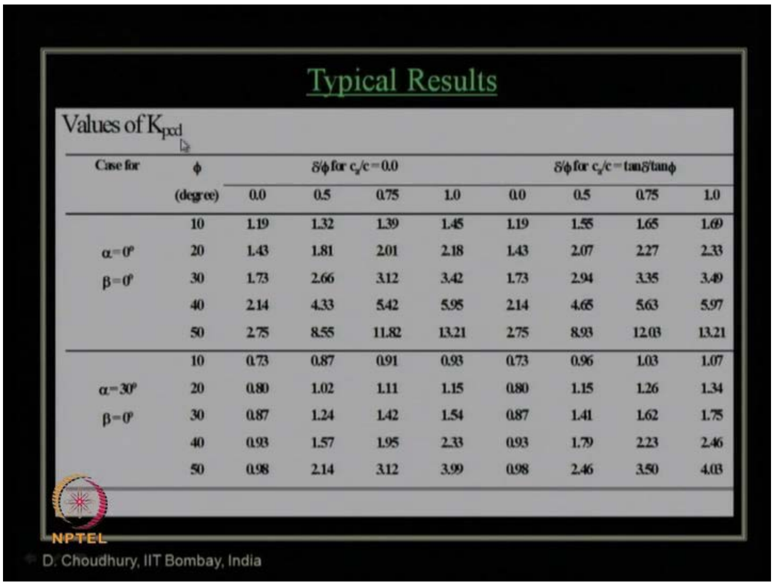

Typical Results

Values of $K_{p\gamma d}$

Case for	ϕ (degree)	δ/ϕ for $c_a/c = 0.0$: 0.0	0.5	0.75	1.0	δ/ϕ for $c_a/c = \tan\delta/\tan\phi$: 0.0	0.5	0.75	1.0
	10	1.19	1.32	1.39	1.45	1.19	1.55	1.65	1.69
$\alpha = 0^\circ$	20	1.43	1.81	2.01	2.18	1.43	2.07	2.27	2.33
$\beta = 0^\circ$	30	1.73	2.66	3.12	3.42	1.73	2.94	3.35	3.49
	40	2.14	4.33	5.42	5.95	2.14	4.65	5.63	5.97
	50	2.75	8.55	11.82	13.21	2.75	8.93	12.03	13.21
	10	0.73	0.87	0.91	0.93	0.73	0.96	1.03	1.07
$\alpha = 30^\circ$	20	0.80	1.02	1.11	1.15	0.80	1.15	1.26	1.34
$\beta = 0^\circ$	30	0.87	1.24	1.42	1.54	0.87	1.41	1.62	1.75
	40	0.93	1.57	1.95	2.33	0.93	1.79	2.23	2.46
	50	0.98	2.14	3.12	3.99	0.98	2.46	3.50	4.03

Similarly for K p q d, and also from this design table one can get for the K p c d depending on what is wall adhesion; whether, it is a 0 adhesion or there is a full adhesion.

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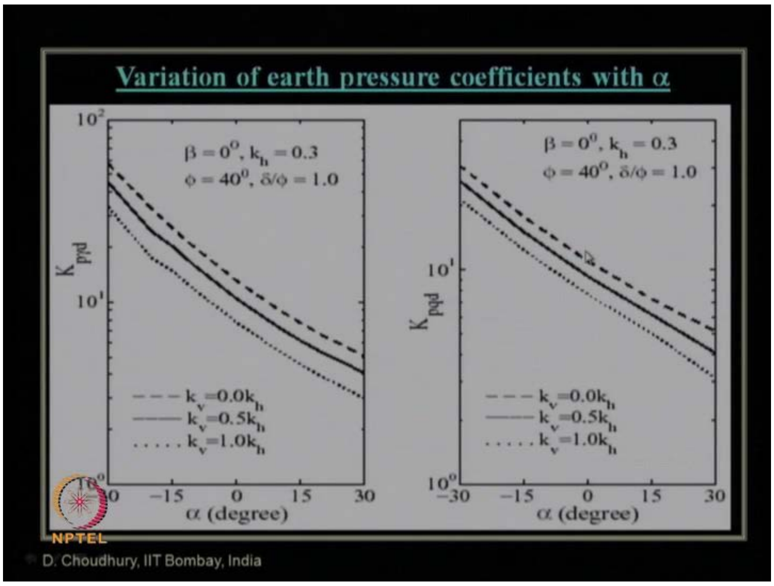

So if it is in between, then one can do a linear interpolation. Now, if one tries to get an optimum design of a retaining wall, so they should know what should be the batter angle.

So, for knowing the batter angle seismic passive resistance should be maximum because passive maximum means you are getting the advantage, whereas for active minimum is, it is advantageous for the design. Right. Whereas, we are getting the active value maximized and passive value minimized.

So, if we see the variation with respect to wall batter angle alpha, you can see the variation of K p gamma d. As alpha changes from negative batter to positive batter, it varies like this. Same, similar trend has been obtained for K p q d.

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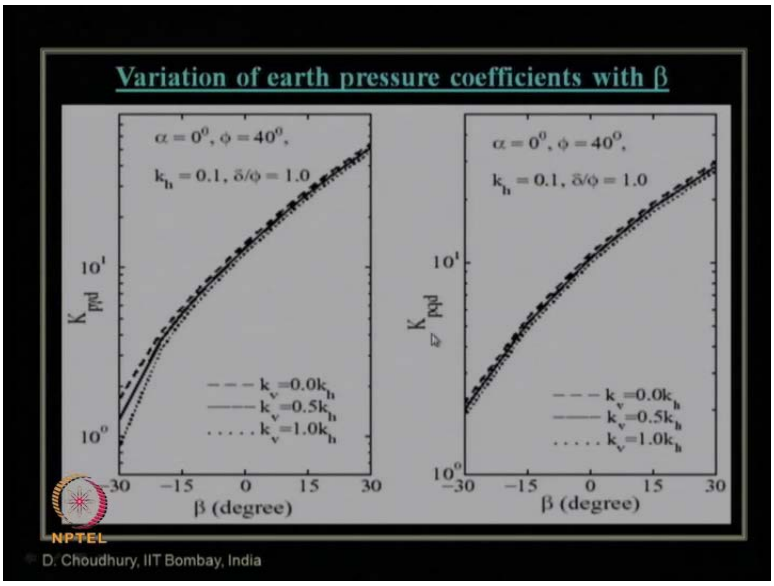

Whereas for ground surface inclination, that is, the surcharge; if it increases for negative to positive the seismic passive earth pressure coefficient keeps on increasing, whether it is K p gamma d or K p q d.

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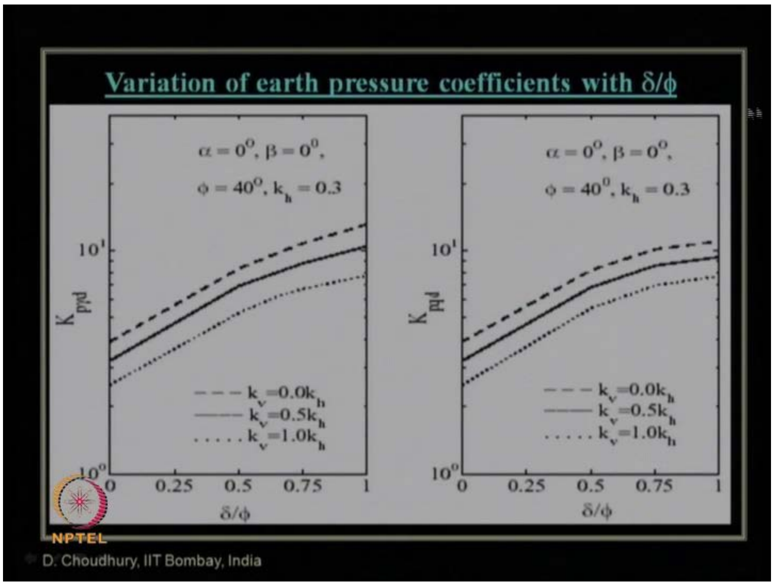

And, with respect to this ratio of wall friction angle to soil friction angle, that is, delta by phi; as we have mentioned, this delta by phi less than equals to one-third means, it can be considered as planer rupture surface. But beyond one-third it should be curved rupture surface. So, if it varies between 0 to 1, the variation of this seismic passive earth pressure coefficients are like this.

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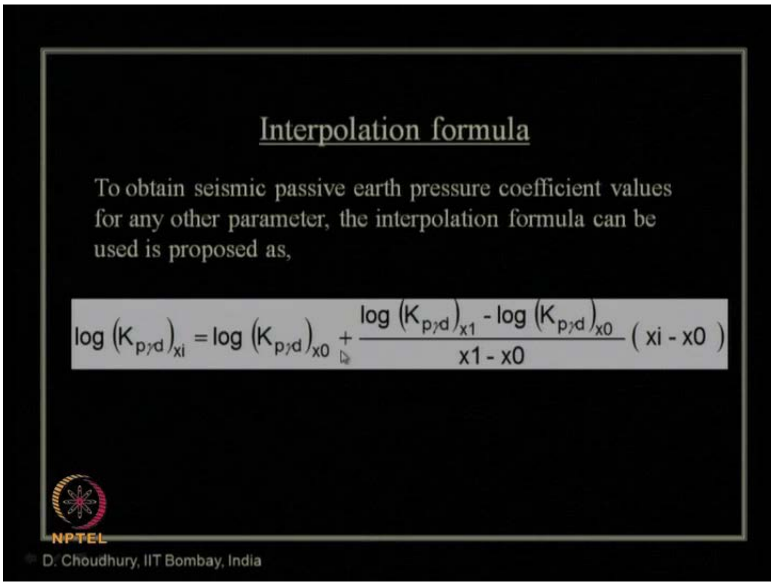

As I said, for intermediate values which are not given in the table or instead of reading directly from the chart, one can get the values from the table and then use this interpolation formula to get intermediate value. That is, we have computed for say k h value; 0.2 g, 0.3 g, 0.4 g like that. If somebody wants for 0.25 g or 0.27 g, they can use this interpolation formula and get the design value easily; because the variation is linear with respect to log scale.

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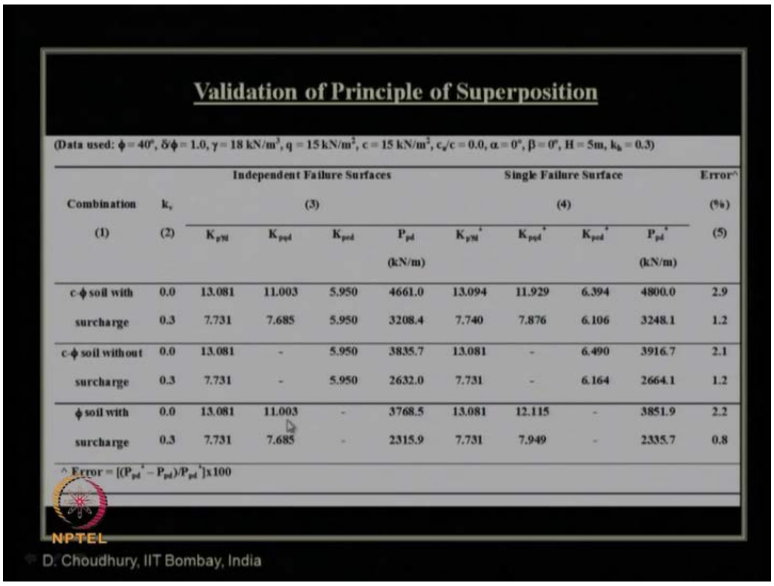

Validation of Principle of Superposition

(Data used: $\phi = 40°$, $\delta/\phi = 1.0$, $\gamma = 18$ kN/m^3, $q = 15$ kN/m^2, $c = 15$ kN/m^2, $c_a/c = 0.0$, $\alpha = 0°$, $\beta = 0°$, $H = 5$m, $k_h = 0.3$)

Combination (1)	k_v (2)	Independent Failure Surfaces (3) $K_{p\gamma d}$	K_{pqd}	K_{pcd}	P_{pd} (kN/m)	Single Failure Surface (4) $K_{p\gamma d}^*$	K_{pqd}^*	K_{pcd}^*	P_{pd}^* (kN/m)	Error^ (%) (5)
c-ϕ soil with surcharge	0.0	13.081	11.003	5.950	4661.0	13.094	11.929	6.394	4800.0	2.9
	0.3	7.731	7.685	5.950	3208.4	7.740	7.876	6.106	3248.1	1.2
c-ϕ soil without surcharge	0.0	13.081	-	5.950	3835.7	13.081	-	6.490	3916.7	2.1
	0.3	7.731	-	5.950	2632.0	7.731	-	6.164	2664.1	1.2
ϕ soil with surcharge	0.0	13.081	11.003	-	3768.5	13.081	12.115	-	3851.9	2.2
	0.3	7.731	7.685	-	2315.9	7.731	7.949	-	2335.7	0.8

^ Error = $[(P_{pd}^* - P_{pd})/P_{pd}^*] \times 100$

Then, validation of superposition also was done. Why the validation of superposition is required? Because you remember we estimated individually the three components; unit weight component, surcharge component and cohesion component. But in reality, it will a single failure surface and not the three individual components will come into picture. So to take care of that, if we take the single failure surface what is the problem? The problem will be… it will be case specific. Case specific in the sense, for a particular data only you will get one set of result; for another change of data you will get another single failure surface.

So, you are not going to get the closed form generalized design solution. So, to get a generalized design solution, independent failure surface is advised to use. So, that is why we try to check what is the error by doing that principle of superposition method by using independent failure surface and the single failure surface for a given set of data. For a given set of data you can see for different combinations like c phi soil with surcharge, c phi soil without surcharge and phi soil with surcharge. That is, all possible cases are taken care of. Percentage error between the actual single failure surface and independent failure surface is very minimum within three percent, which is acceptable range. And not only that, that error is on the safer side. Why safer side? If you look at the value of these total Pp d and Pp d star in the case of single failure surface, in Pp d if using independent failure surfaces is the absolute minimum one and for passive case of design we want the absolute minimum one. So, this error is on the safer side of the error and that too this error is on the, within the range of within three percent. So, this independent failure surface approach or the principle of superposition is, can be well accepted in the design.

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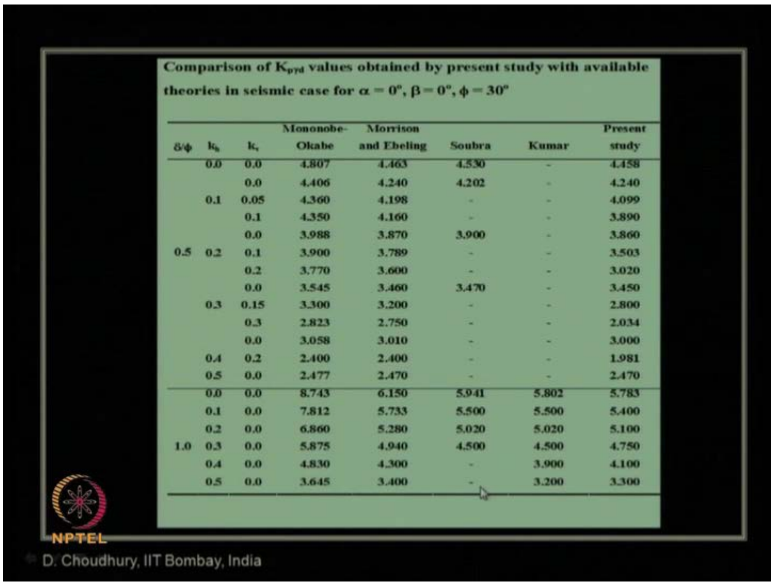

Comparison of $K_{p\gamma d}$ values obtained by present study with available theories in seismic case for $\alpha = 0°$, $\beta = 0°$, $\phi = 30°$

δ/ϕ	k_h	k_v	Mononobe-Okabe	Morrison and Ebeling	Soubra	Kumar	Present study
	0.0	0.0	4.807	4.463	4.530	-	4.458
		0.0	4.406	4.240	4.202	-	4.240
	0.1	0.05	4.360	4.198	-	-	4.099
		0.1	4.350	4.160	-	-	3.890
		0.0	3.988	3.870	3.900	-	3.860
0.5	0.2	0.1	3.900	3.789	-	-	3.503
		0.2	3.770	3.600	-	-	3.020
		0.0	3.545	3.460	3.470	-	3.450
	0.3	0.15	3.300	3.200	-	-	2.800
		0.3	2.823	2.750	-	-	2.034
		0.0	3.058	3.010	-	-	3.000
	0.4	0.2	2.400	2.400	-	-	1.981
	0.5	0.0	2.477	2.470	-	-	2.470
	0.0	0.0	8.743	6.150	5.941	5.802	5.783
	0.1	0.0	7.812	5.733	5.500	5.500	5.400
	0.2	0.0	6.860	5.280	5.020	5.020	5.100
1.0	0.3	0.0	5.875	4.940	4.500	4.500	4.750
	0.4	0.0	4.830	4.300	-	3.900	4.100
	0.5	0.0	3.645	3.400	-	3.200	3.300

The comparison of the our proposed value of this K p gamma d coefficients of seismic passive earth pressure with respect to other researchers are shown over here. Obviously, our values are far away from the Mononobe-Okabe because of the reason, Mononobe-Okabe considered the planer rupture surface for passive case which seriously overestimates the value right. Whereas, present study gives the critical value of the design value or the minimum value compared to Mononobe-Okabe. And compared to other researchers also the optimization has been shown over here.

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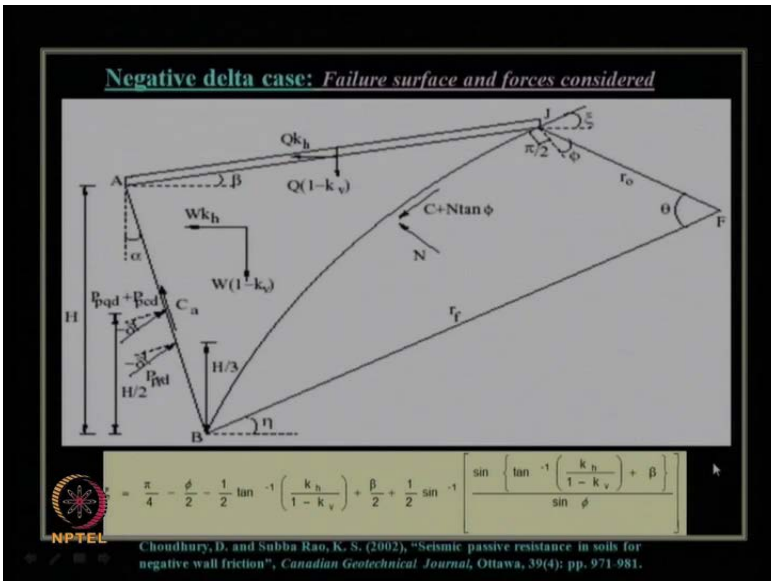

Similarly, for negative delta case also we have described in previous lecture, which is available in the publication in the "Canadian Geotechnical journal", "Choudhury and Subba Rao 2002".This is the volume and page number. That, for a negative friction angle case of passive earth pressure, this is the failure surface. Curved failure surface of logarithmic surface was considered and the design values of this passive earth pressure coefficients are obtained.

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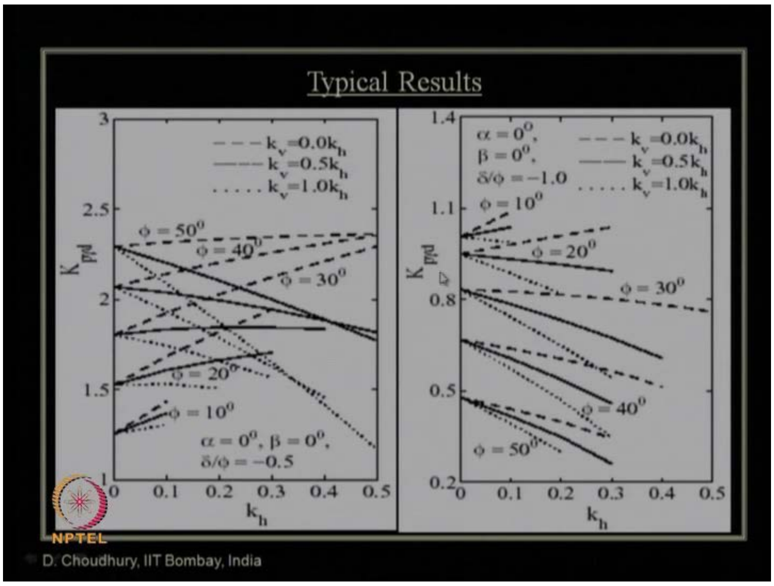

These are the typical design charts for this passive earth pressure coefficients in terms of K p gamma d, K p q d.

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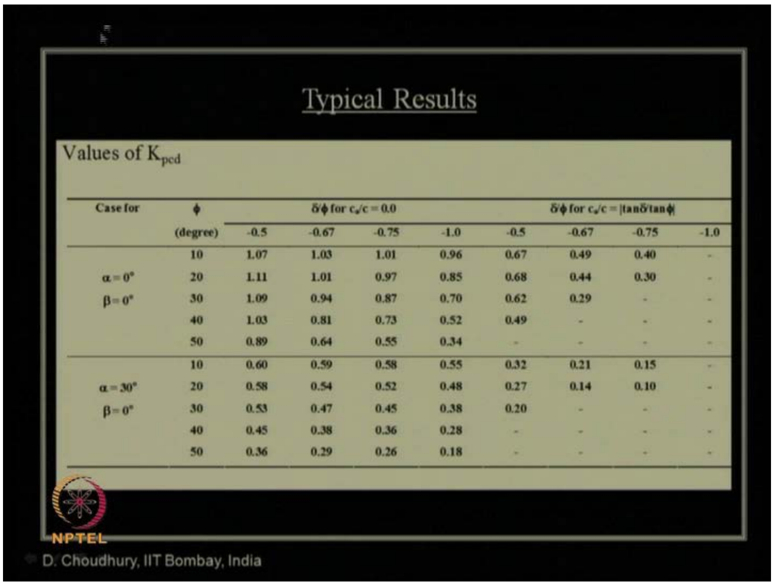

Typical Results

Values of K_{pcd}

Case for	ϕ	δ/ϕ for $c_a/c = 0.0$				δ/ϕ for $c_a/c = \lvert\tan\delta/\tan\phi\rvert$			
	(degree)	-0.5	-0.67	-0.75	-1.0	-0.5	-0.67	-0.75	-1.0
$\alpha = 0°$, $\beta = 0°$	10	1.07	1.03	1.01	0.96	0.67	0.49	0.40	-
	20	1.11	1.01	0.97	0.85	0.68	0.44	0.30	-
	30	1.09	0.94	0.87	0.70	0.62	0.29	-	-
	40	1.03	0.81	0.73	0.52	0.49	-	-	-
	50	0.89	0.64	0.55	0.34	-	-	-	-
$\alpha = 30°$, $\beta = 0°$	10	0.60	0.59	0.58	0.55	0.32	0.21	0.15	-
	20	0.58	0.54	0.52	0.48	0.27	0.14	0.10	-
	30	0.53	0.47	0.45	0.38	0.20	-	-	-
	40	0.45	0.38	0.36	0.28	-	-	-	-
	50	0.36	0.29	0.26	0.18	-	-	-	-

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And, these are the table for K p c d for different ranges of wall adhesion. So, any wall adhesion in between, one can easily interpolate using that interpolation formula.

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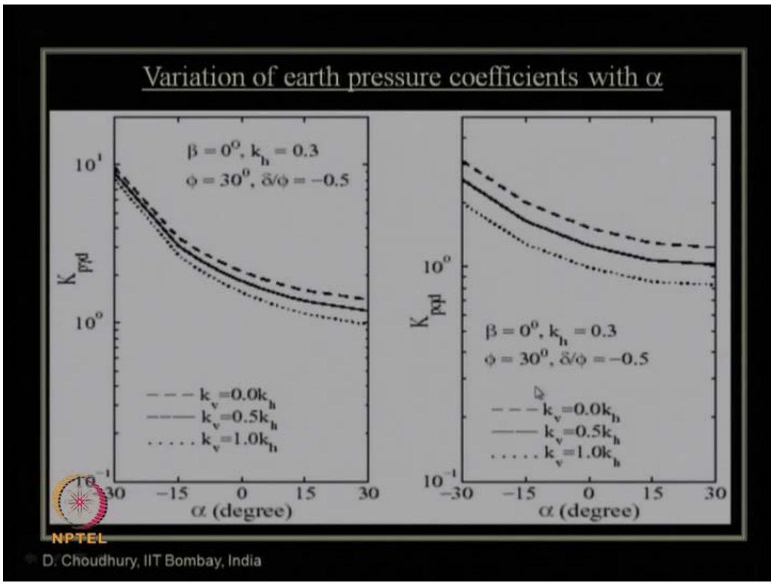

We have also seen the variation of alpha; that is wall batter. That is, how it effects on this seismic earth pressure coefficients.

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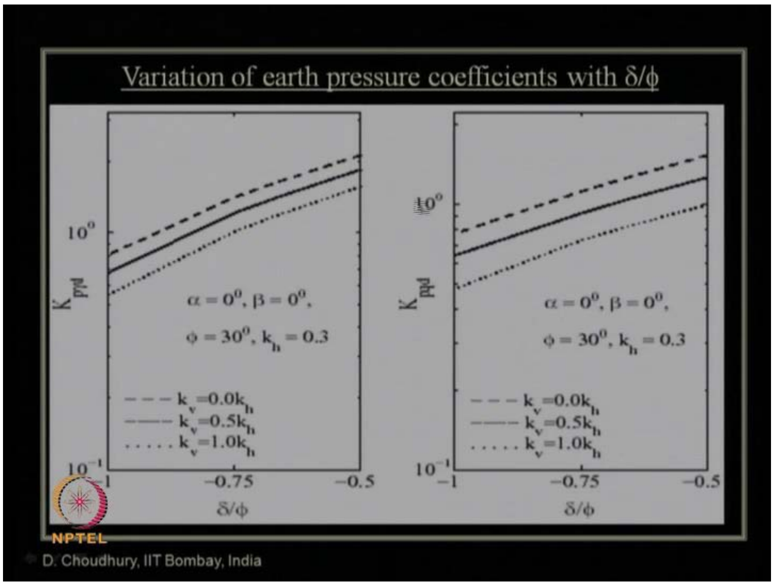

Also the variation of delta by phi; how it effects in the design values of this seismic passive earth pressure coefficients.

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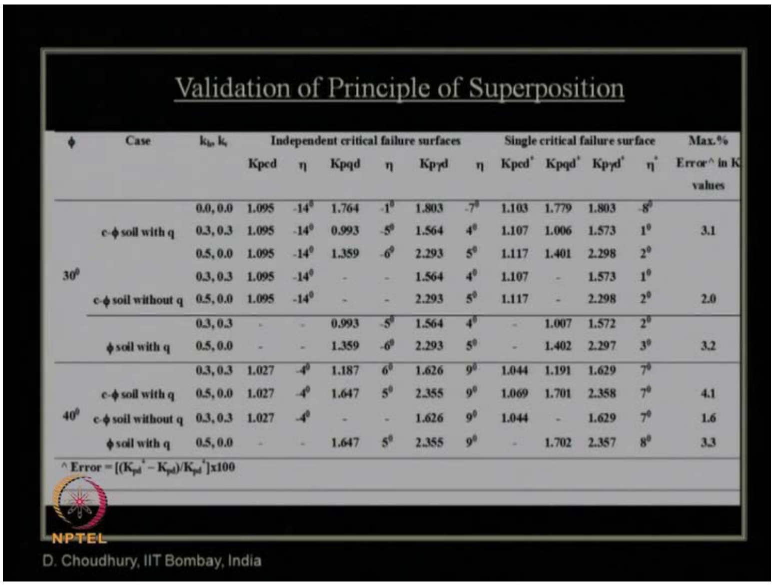

ϕ	Case	k_h, k_v	Independent critical failure surfaces						Single critical failure surface				Max.%
			Kpcd	η	Kpqd	η	Kpγd	η	Kpcd*	Kpqd*	Kpγd*	η*	Error^ in K values
30^0	c-ϕ soil with q	0.0, 0.0	1.095	-14^0	1.764	-1^0	1.803	-7^0	1.103	1.779	1.803	-8^0	
		0.3, 0.3	1.095	-14^0	0.993	-5^0	1.564	4^0	1.107	1.006	1.573	1^0	3.1
		0.5, 0.0	1.095	-14^0	1.359	-6^0	2.293	5^0	1.117	1.401	2.298	2^0	
	c-ϕ soil without q	0.3, 0.3	1.095	-14^0	-	-	1.564	4^0	1.107	-	1.573	1^0	
		0.5, 0.0	1.095	-14^0	-	-	2.293	5^0	1.117	-	2.298	2^0	2.0
	ϕ soil with q	0.3, 0.3	-	-	0.993	-5^0	1.564	4^0	-	1.007	1.572	2^0	
		0.5, 0.0	-	-	1.359	-6^0	2.293	5^0	-	1.402	2.297	3^0	3.2
40^0	c-ϕ soil with q	0.3, 0.3	1.027	-4^0	1.187	6^0	1.626	9^0	1.044	1.191	1.629	7^0	
		0.5, 0.0	1.027	-4^0	1.647	5^0	2.355	9^0	1.069	1.701	2.358	7^0	4.1
	c-ϕ soil without q	0.3, 0.3	1.027	-4^0	-	-	1.626	9^0	1.044	-	1.629	7^0	1.6
	ϕ soil with q	0.5, 0.0	-	-	1.647	5^0	2.355	9^0	-	1.702	2.357	8^0	3.3

^ Error = $[(K_{pd}^* - K_{pd})/K_{pd}^*]$x100

Here also, the principle of superposition has been checked using independent as well as single failure surface for a chosen data. And it has been found that the maximum error here also is within five percent only for 0.1 percent. And that too, here as well the error is on the safer side. That is, we are estimating by using independent failure surface; the minimum value of the passive earth pressure.

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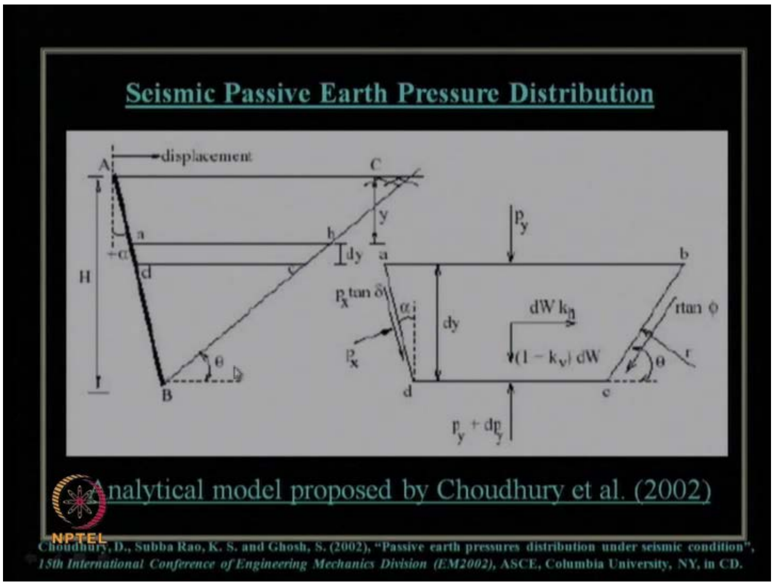

Then in the previous lecture, we have also discussed how to estimate the point of application of the total passive resistance acting on a wall under this seismic condition using this horizontal slice approach. So, for individual slice these are the forces acting. One assumption is that, these two plane of this slices are considered as the principle plane. So, there is no shear forces acting. So, this is the publication available in "Choudhury, Subba Rao and Ghosh" in the proceedings of the fifteenth International conference of Engineering Mechanics division, which is EM2002 of ASCE.

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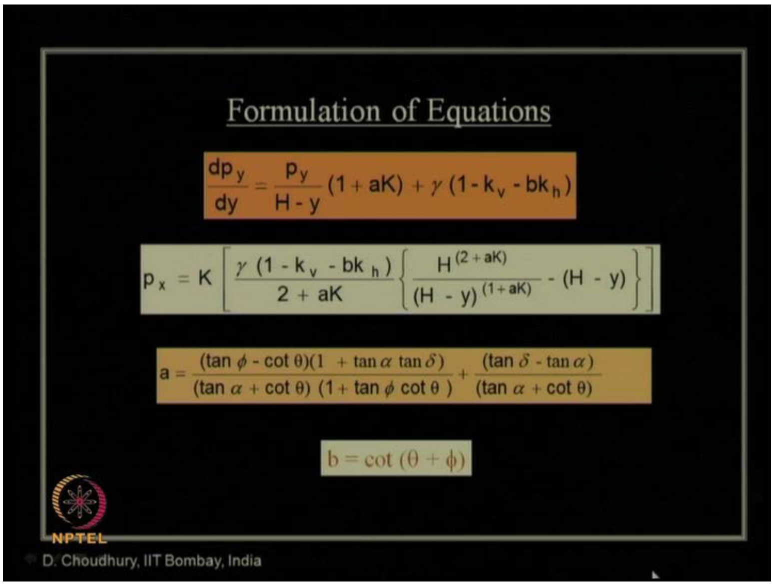

These are the formulation of the equation of the earth pressure. And these are final seismic passive earth coefficient. As you can see, the assumption was the planer rupture surface. So, the values of this seismic passive earth pressure coefficients what we are getting using this approach and Mononobe-Okabe's equation should remain same. Right. That is one of the validity.

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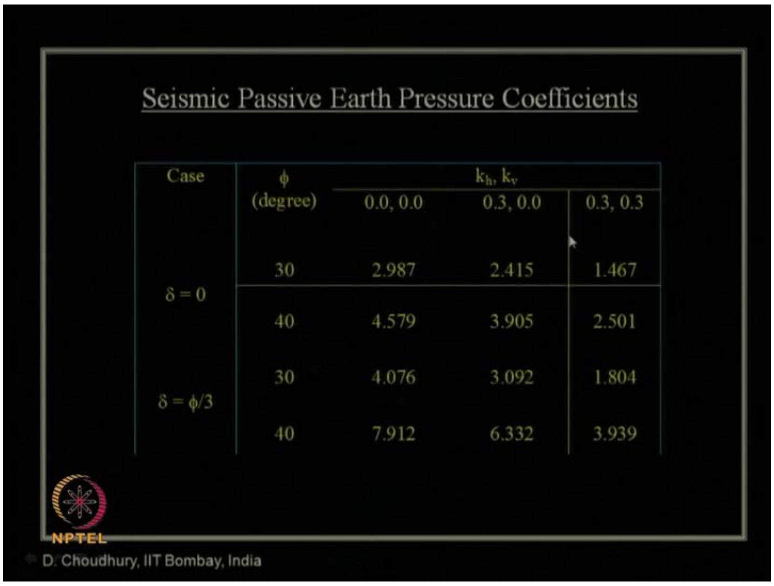

Case	ϕ (degree)	k_h, k_v		
		0.0, 0.0	0.3, 0.0	0.3, 0.3
$\delta = 0$	30	2.987	2.415	1.467
	40	4.579	3.905	2.501
$\delta = \phi/3$	30	4.076	3.092	1.804
	40	7.912	6.332	3.939

So they remain same, but what is the advantage of this method of slice which you are not getting in the method in the Mononobe Okabe method? That the point of application was not known.

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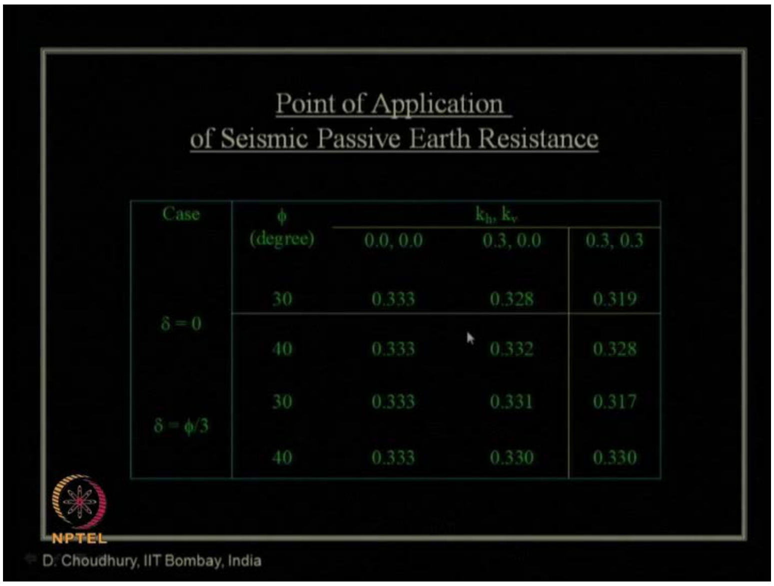

Case	ϕ (degree)	k_h, k_v: 0.0, 0.0	k_h, k_v: 0.3, 0.0	k_h, k_v: 0.3, 0.3
$\delta = 0$	30	0.333	0.328	0.319
	40	0.333	0.332	0.328
$\delta = \phi/3$	30	0.333	0.331	0.317
	40	0.333	0.330	0.330

In this case, you can see the point of application for static case. It is always one-third from the height which is all known to us. But in seismic case it is not at one-third, but some other different height depending on the combinations of k h, k v.

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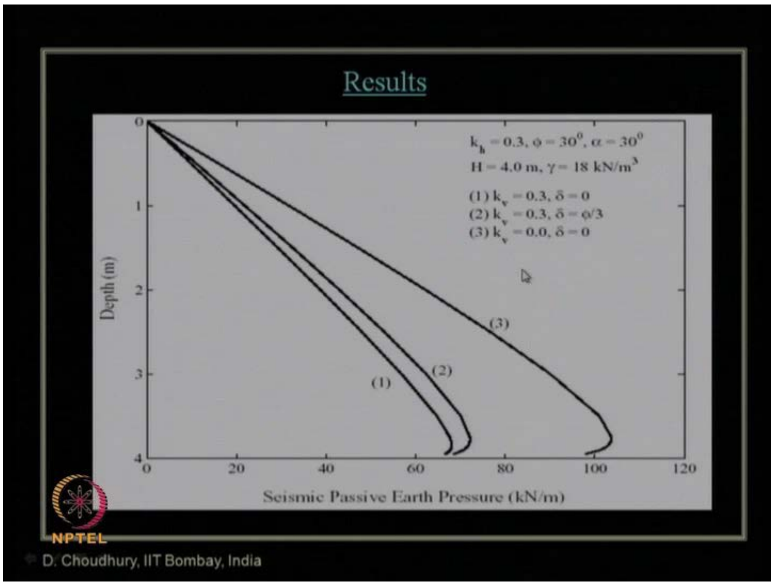

And, other combinations of inclinations of the wall; you can see there is a non-linearity involved closed to the bottom of the or base of the wall.

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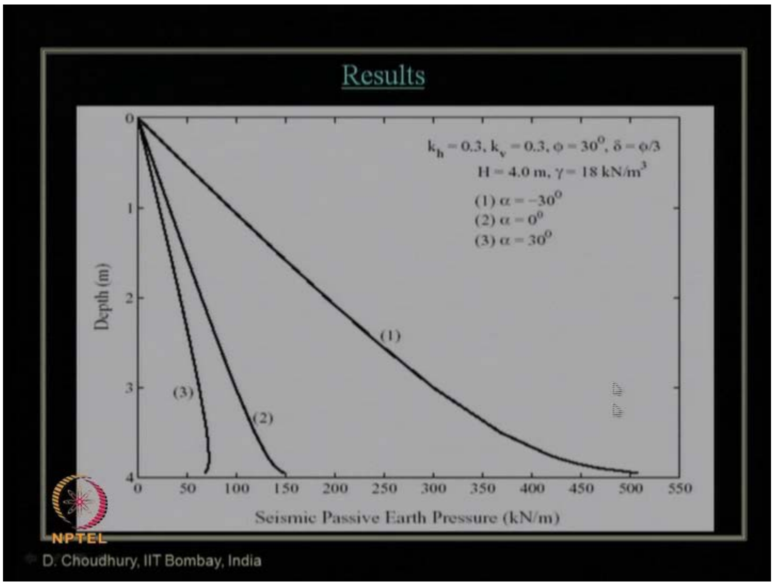

And that non-linearity depends whether it is convex upward or concave upward, depending on the inclination of the wall.

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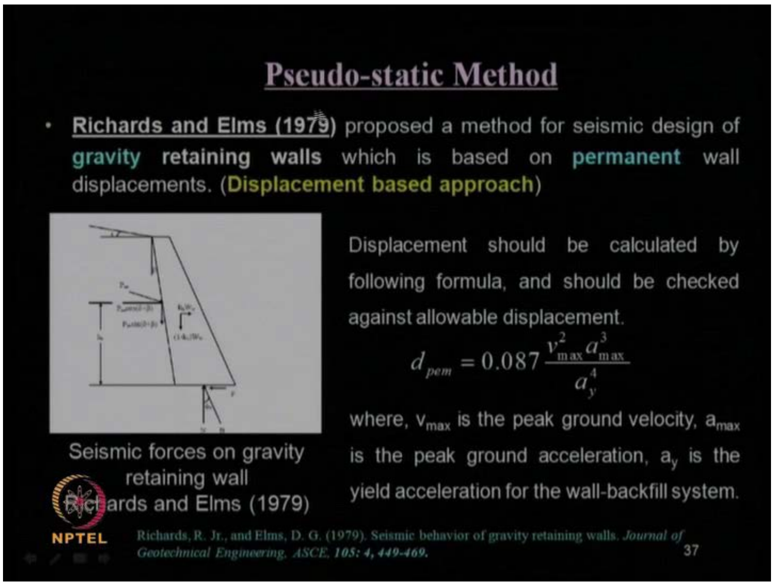

Then, in the previous lecture we also discussed about the basic displacement based approach, which was proposed by Richard and Elms by considering the permanent displacement of the wall, considering the sliding of the wall using the pseudo static approach; which is available in this ASCE journal, paper journal of Geotechnical Engineering by Richard and Elms in 1979.

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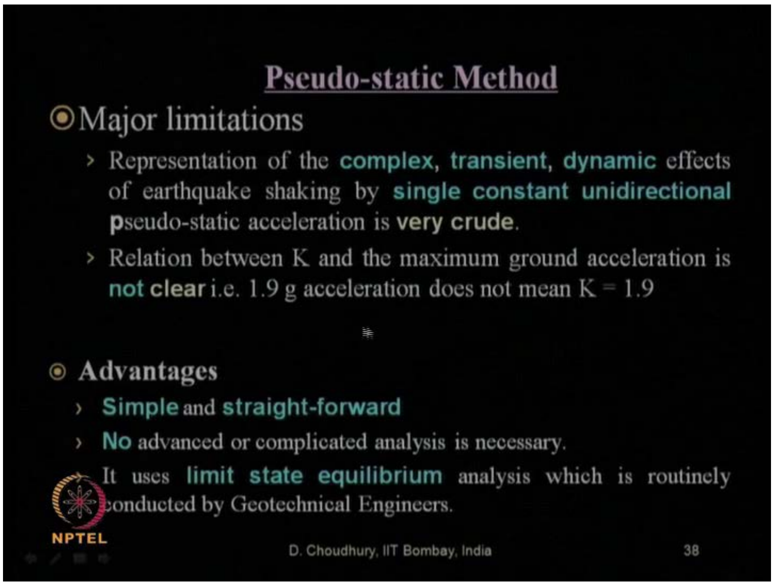

We also discussed in our previous lecture, what are the major limitations about pseudo static approach like, it is very crude to approximate the actual complex nature of the dynamic problem and transient nature of the earthquake shaking into a single constant unidirectional coefficient or value in the analysis. But why still people use it? Because of its simplicity and straightforwardness, and no advance complex analysis is required. So, that is why this method is still popular among engineers.

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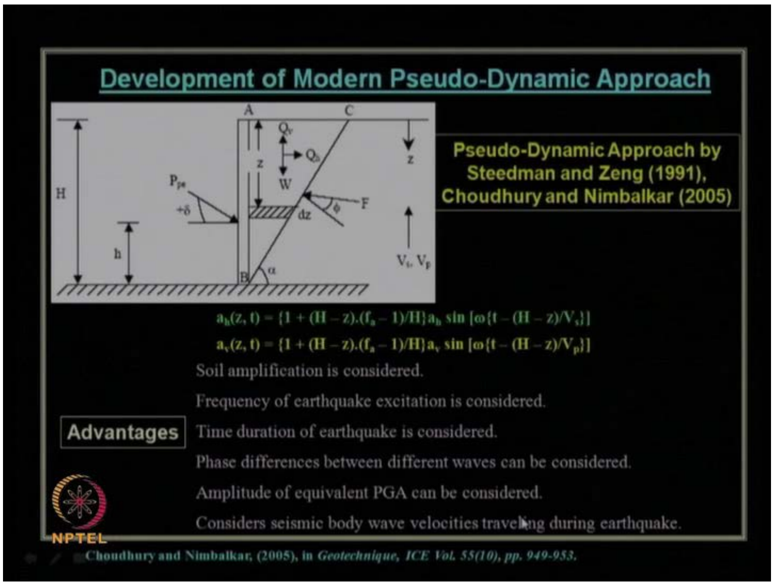

To overcome all these limitations of pseudo static method, modern pseudo dynamic approach was proposed. As I said Steedman and Zeng originally proposed considering only the horizontal seismic inertia force and the shear wave velocity. Then later on my first Ph.D. student Dr Sanjay Nimbalkar from his Ph.D. thesis work, he has given a generalized solution of this pseudo dynamic approach considering both horizontal as well as vertical seismic inertia force and both shear wave velocity and primary wave velocity, which are coming; including the soil amplification factor, duration of earthquake and frequency of earthquake motion.

So, these are the advantages which are not available in the pseudo static approach. So, details about this analysis is available in this journal paper “Choudhury and Nimbalkar2005” in journal “Geotechnique” published by “Institute of Civil Engineers”, London, United Kingdom. This is the volume number and page number.

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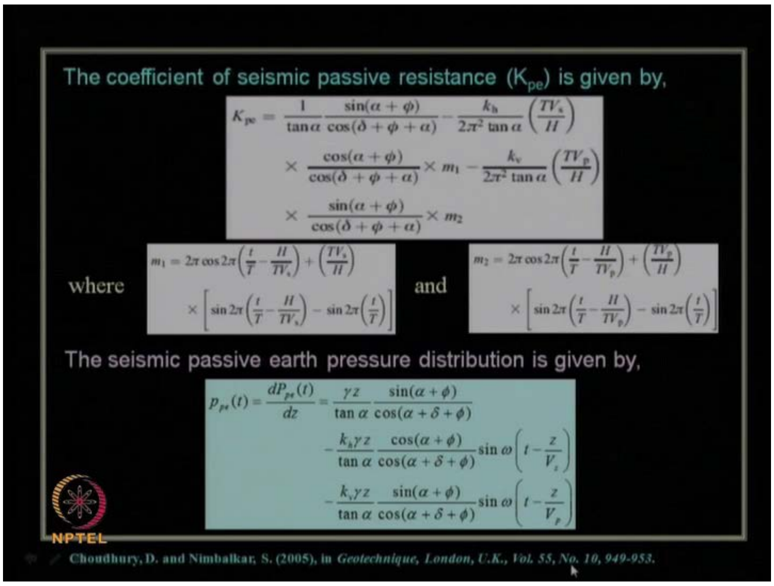

And, we had proposed this closed form design solution or the designer’s equation, which is necessary to design an seismic passive earth pressure. In case of retaining wall, the seismic passive earth pressure coefficient expression K p e which needs to be optimized with respect to this alpha failure angle as well as this t by T ratio; that is, duration of earthquake and the period of earthquake that needs to be optimized to get the minimum value of this K p e.

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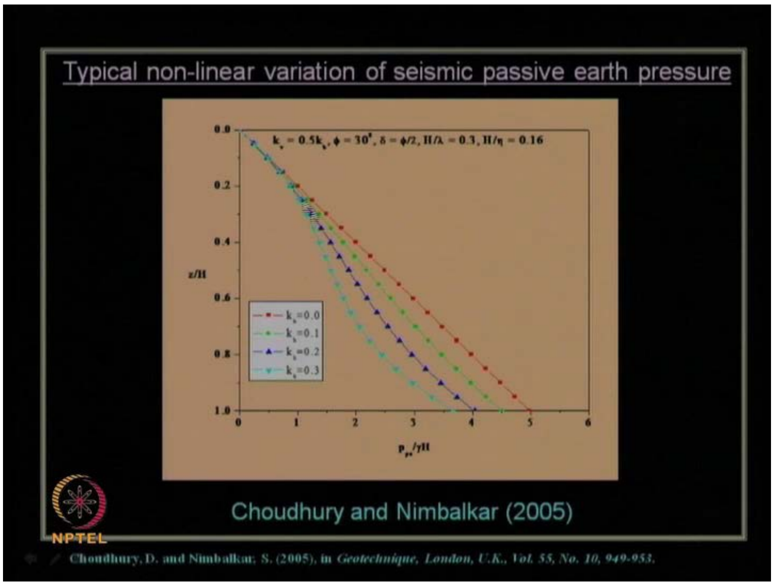

And another advantage in this case of pseudo dynamic, we are getting the distribution of the passive earth pressure with respect to the wall which can be seen from this result that, this red line is the static one as seismicity increases as expected, the design value of passive earth pressure is decreasing and non-linearity of it is increasing with increase in the seismicity.

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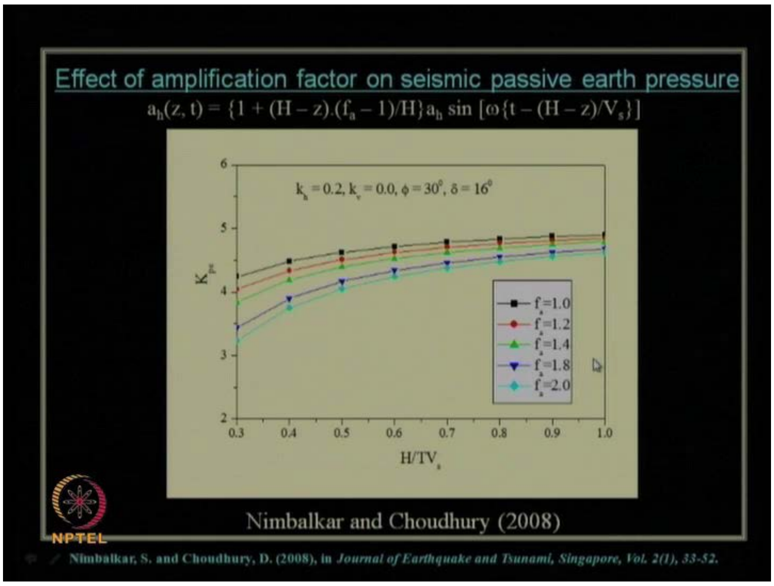

And, the effect of this soil amplification factor can be seen on the seismic passive earth pressure coefficient. As the soil amplification factor increases, there is a decrease, significant decrease in this seismic passive earth pressure coefficient value which needs to be considered in the design; which is available in journal paper “Nimbalkar and Choudhury 2008” in the journal of “Earthquake and Tsunami”.

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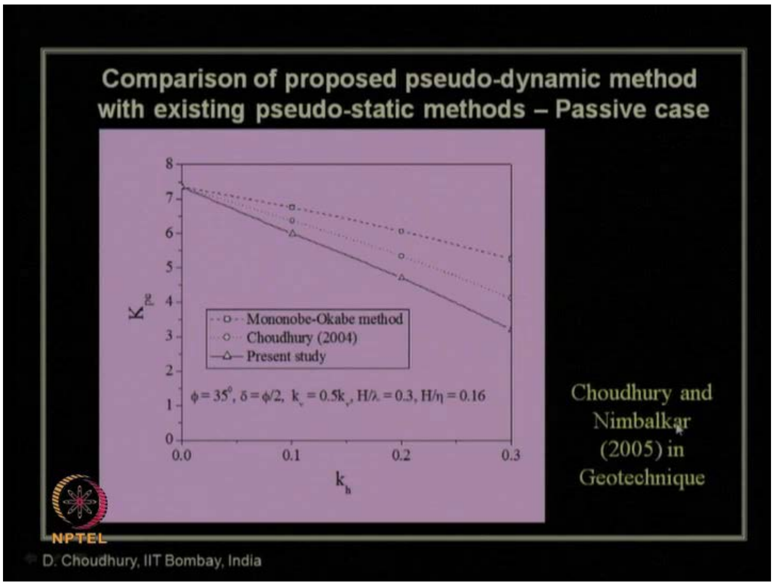

These are the comparison of the proposed pseudo dynamic method of Choudhury and Nimbalkar with respect to the pseudo static method of Mononobe-Okabe and Choudhury two thousand four, which is available in “Geotechnique”. You can see the pseudo dynamic method gives the critical value of the design value; the minimum value of the seismic passive earth pressure coefficient for any given value of k h as shown over here.

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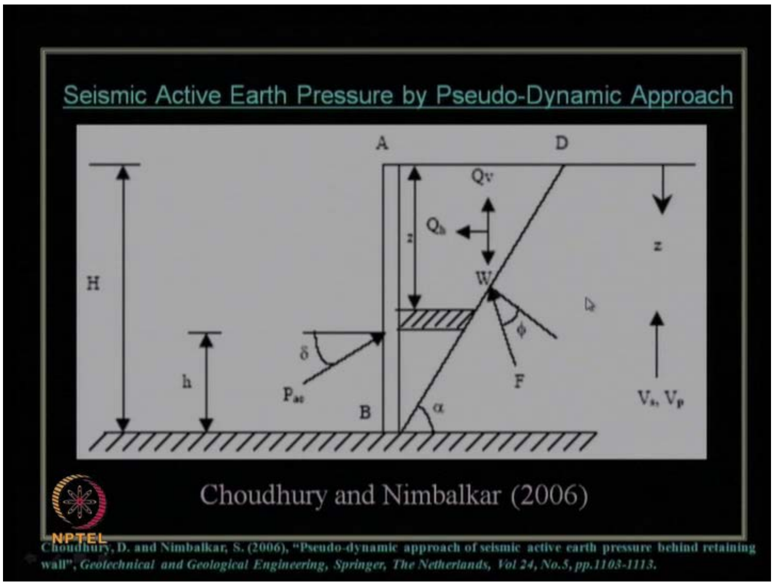

Then, we have seen the active earth pressure, seismic active earth pressure determination by using the pseudo dynamic approach as given by "Choudhury and Nimbalkar 2006".This is the journal paper, "Geotechnical and Geological Engineering", Springer publication in 2006.

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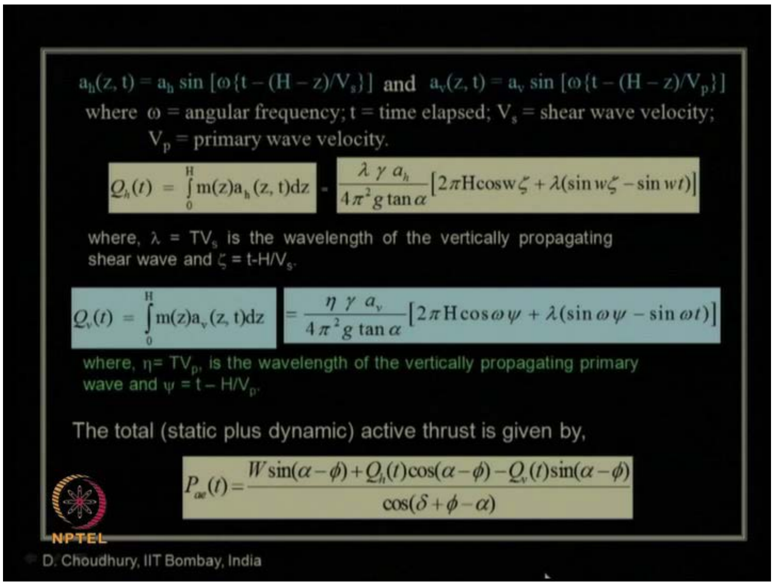

These are the formulation or estimation of the seismic horizontal and vertical inertia forces. That is, multiplying that infinitesimal soil mass with respect to the acceleration and integrating that over the entire depth of the wall for that infinitesimal soil mass which is shown over here. And then doing the equilibrium of all the forces, the seismic active earth pressure Pa e can be obtained like this.

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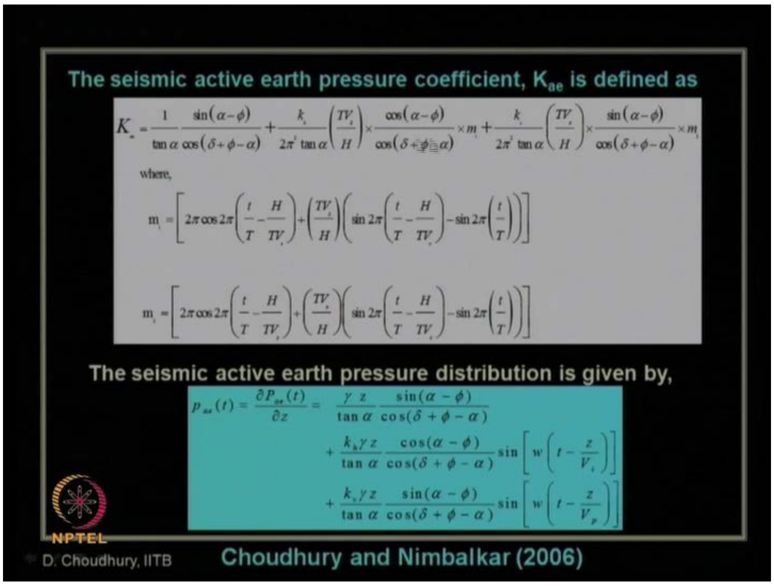

And, seismic active earth pressure coefficient K a e also in the closed form solution or the design solution can be given like this. Also, its distribution can be obtained using this expression.

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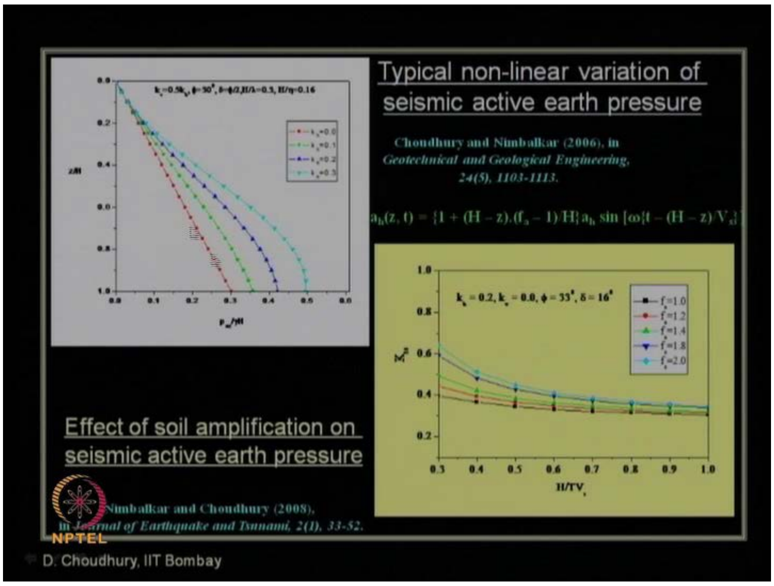

Which can be seen from this result; one typical result for this given data set has been shown over here. This red color shows the static value of the earth pressure. And as the seismicity increases, the seismic earth pressure in the active state should increase. That is, the design value and non-linearity of it also increases. That is, its point of application keeps towards upward from that one-third of from the base of the wall, which is generally considered for the static case. And also the effect of soil amplification factor for the seismic active earth pressure also shown over here. As amplification increases, you can see there is a significant increase in the value of seismic active earth pressure coefficient which needs to be considered in the design.

Now, let us come to the experimental validation of this pseudo dynamic method; because this pseudo dynamic method is a recent method. It needs to be validated using the experimental results available. So for that the Geotechnical Dynamic Centrifuge facility which is available at university of California Davis in USA.

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The results obtained by research, other researchers were used. They did an analysis for this BART. BART is nothing but bay area rapid transit or a metro railway system. For underground tunnel, they need to redesign it under different earthquake condition; latest earthquake condition.

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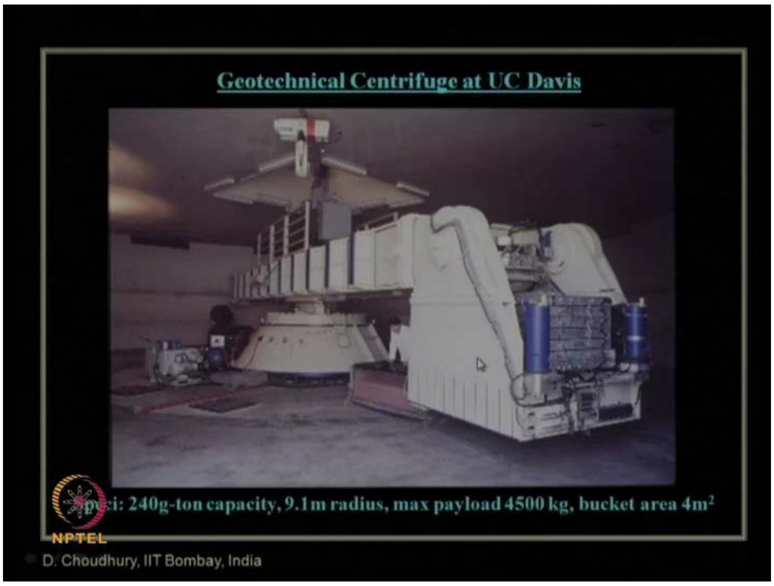

That was the project which they have done using this largest Geotechnical centrifuge in the world, which is available at university of California at Davis campus. This is the capacity240 g ton capacity. And that centrifuge can do the dynamic testing; because in the shaker, during shaking they have the one shaking table which is installed in the shaker. That is, during the flight it can simulate the vibration and these are the dimension; 9.1 meter of the radius of the centrifuge, other payload and bucket area is given over here.

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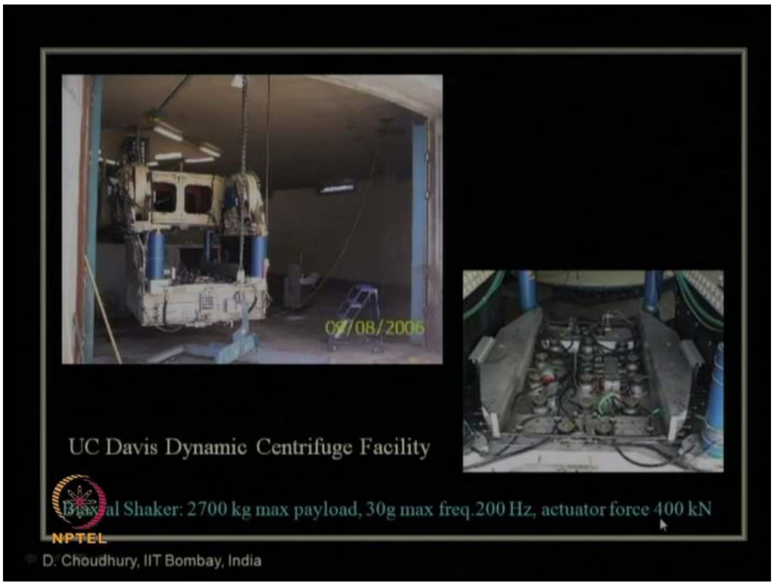

You can see, it can go up to 30 g maximum frequency of 200 hertz with actuator force of 400 Newton. This is a biaxial shaker which is implanted on this box of this centrifuge container position, where it can be kept.

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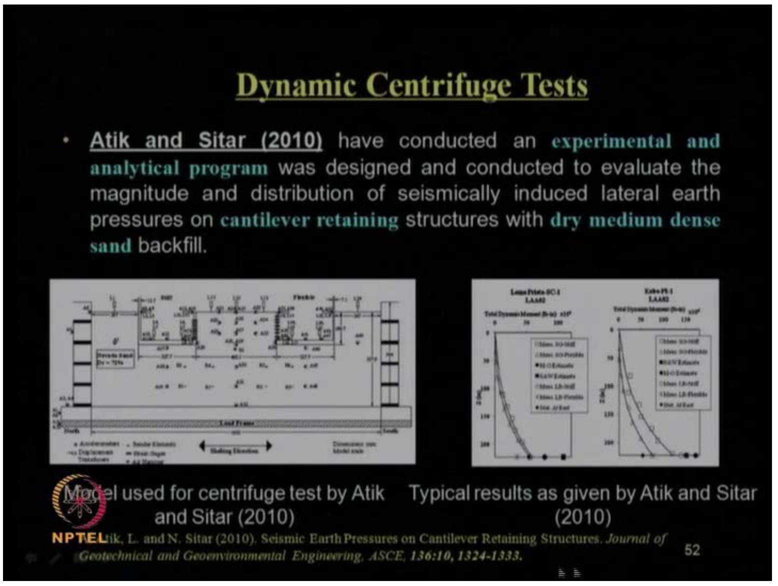

The researchers like Atik and sitar who have published their data in 2010 in ASCE journal of Geotechnical and Geoenvironmental Engineering. They have given all the results of this BART tunnel, which they have analyzed in this dynamic centrifuge using different input earthquake motion. And they have compared their dynamic earth pressure. Under the active condition of earth pressure, what are the earth pressure values they are getting under different earthquake excitations.

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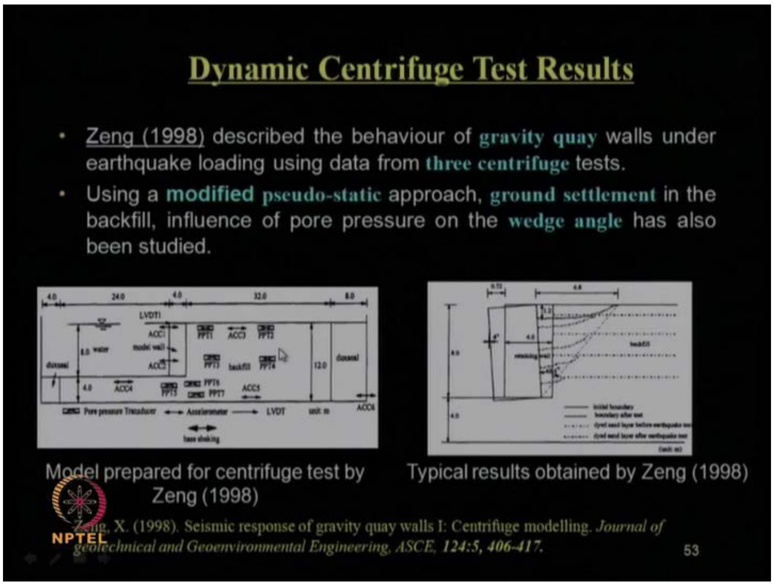

The similar test, dynamic centrifuge test on the retaining wall was also done by Zeng in 1998.It is a Ph.D. thesis work of professor Zeng at university; Cambridge University at UK under the supervision of Professor Steedman.

So, in the Zeng's thesis you can see this is the use of the dynamic centrifuge facility of Cambridge University in UK. They also did the test. The results are available in this journal of "Geotechnical and Geoenvironmental Engineering of ASCE 1998.

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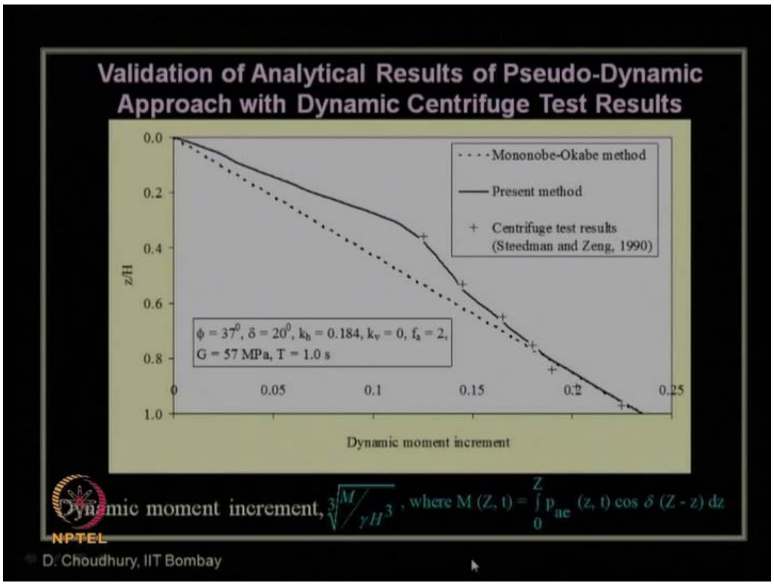

And, these are the results, which shows a non-dimensional depth in the y axis and dynamic moment increment. What is dynamic moment increment? This is how the dynamic moment increment is expressed. That is, the seismic active earth pressure which is a function of z and t now, integrated over the entire height and get the moment; because for moment, you need to multiply this earth pressure with respect to the lever arm or the distance from the base.

So, that way you can see this dotted line shows the Mononobe-Okabe's pseudo static method, which is the linear one. As we all know this will show always a linear variation. The centrifuge test result, dynamic centrifuge test results of Steedman and Zeng of 1990; these are the points plus sign; whereas the present pseudo dynamic method, this is the non-linear variation. This line in terms of dynamic moment increment is shown over here.

So, the pseudo dynamic approach is very closely matching with the exact observation what has been found in this dynamic centrifuge test, which is not the case with the conventional pseudo static approach. This also says the advantage or the necessity of using the pseudo dynamic approach compared to the pseudo static approach.

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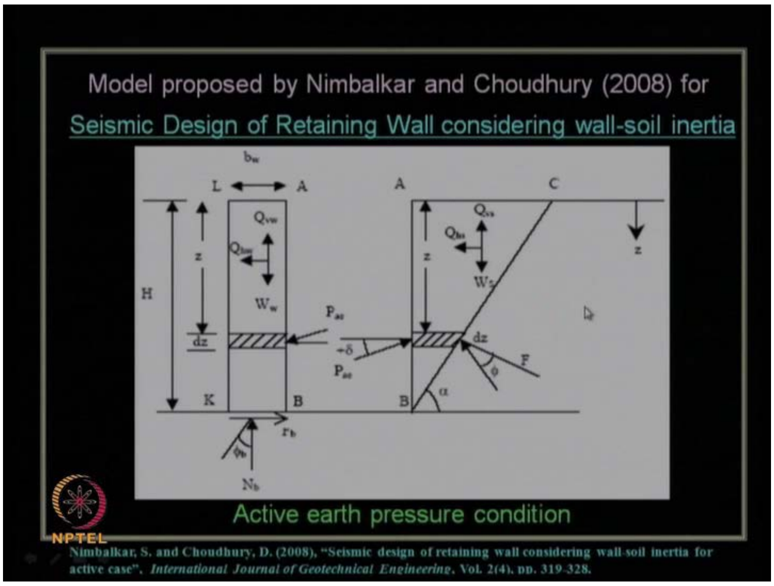

Now to design retaining wall section, as I have already mentioned not only this force based method we should adopt; that is, we should not be happy only by knowing this total earth pressure and their point of application. But also, we should know how much displacement of this wall, etcetera is occurring. For that, we can consider either sliding or the rotational movement of the wall.

So, how to consider that? Let us see over here. This is from the work published by "Nimbalkar and Choudhury 2008".This is the detail of the paper. It is published in International journal of "Geotechnical Engineering in 2008". Seismic design of retaining wall, considering wall and soil inertia; in all the previous problem we were considering the seismic active earth pressure taking into consideration of the seismicity within the failure zone of the soil mass only. But the same inertia or similar type of seismic inertia will act on the wall section also. So, that also needs to be considered.

And when we are considering that, similar to the method of Richard and Elms what I have explained earlier for the pseudo static case, now the pseudo dynamic case has been applied for this wall as well as soil considering this infinitesimal wall element here and soil element here, considering the sliding of the block. So, if it tends to slide in this direction, in this direction the corresponding shear forces, etcetera will come into picture as shown over here.

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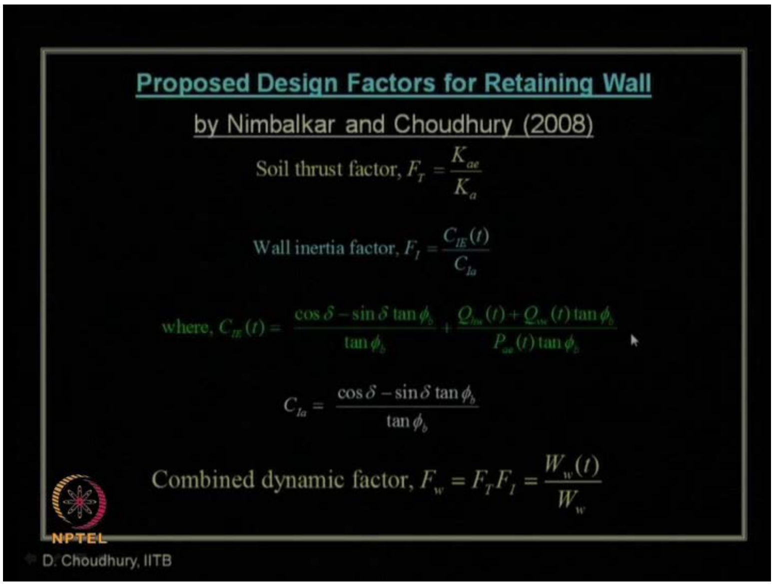

From the analysis, it has been proposed one three different factors were proposed. One is soil thrust factor, which is nothing but ratio of that seismic active earth pressure to the static active earth pressure; another is wall inertia factor, which is given by this C I e which is the function of time by C I a which is a static function.

So, the expressions for C I et and C I a is given. So, C I a is for the static case. This phi b is nothing but the tan of phi b is nothing but this interface friction between the base of the wall and the foundation soil. And for the dynamic case it should take care of the wall inertia, which is also acting on the wall in the horizontal and vertical directions. Can you see over here? So, these are the wall inertia factors. And the combined dynamic factor F w is defined as the multiplication of this soil thrust factor and wall inertia factor.

So, for we talked about soil thrust factor, now only we are introducing this wall inertia factor. Can you see that? So, the total dynamic or combined dynamic factor is expressed as ratio of W w t by W w. What does it mean? W w is nothing but weight of this wall required for the stability of the wall under sliding mode of failure under static condition. And this W w t refers to the weight of the wall under seismic loading condition.

What does it mean? Suppose, if we are able to give this design values or design chart for this soil thrust factor and wall inertia factor, automatically we will get the design chart for combined dynamic factor. Once you know the combined dynamic factor for a particular seismicity level and soil input parameters, you already know how much weight of the wall you should provide for static stability with respect to sliding mode of failure. Now knowing this F w and W w, you can automatically find out what should be your weight of the wall under seismic condition. Clear.

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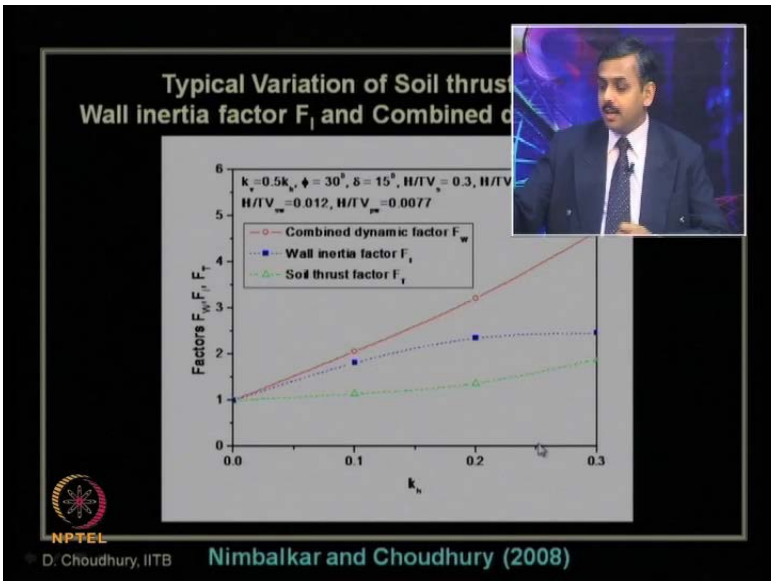

Let me clarify it once again. If we look at this variation of this proposed design factor what we have proposed here in this paper, this red color line gives us the combined dynamic factor F w.

So, suppose somebody wants to design the wall at aseismic zone, where the k h value, the input value can be taken as 0.2 g. And k v value should be taken. Let us say half of k h, so 0.1 g. So, for that combination the value of F w comes out to be about three; little higher than three, three point something as you can see from this point. Right. That means, if you want to design the wall in this section what you should do? One should know what is the wall weight should be provided. How you can get the wall weight? Based on the stability, sliding stability ratio we use the factor safety of 1.5.The same 1.5 you can use, and provide the section of the wall, some width you can provide, height of the wall will be already given to you; because it generally the height of the retaining wall based on your usage of the retaining wall.

So width, etcetera, you can play with and provide a section of the wall, which will give the value of W w under static condition. Once you know that W w using that, F w factor you can get what should be the weight under seismic condition. So, that much weight you should provide to the gravity retaining wall to withstand an earthquake of magnitude of 0.2 g. That is what it means. Clear. So, these are the design steps what one should follow.

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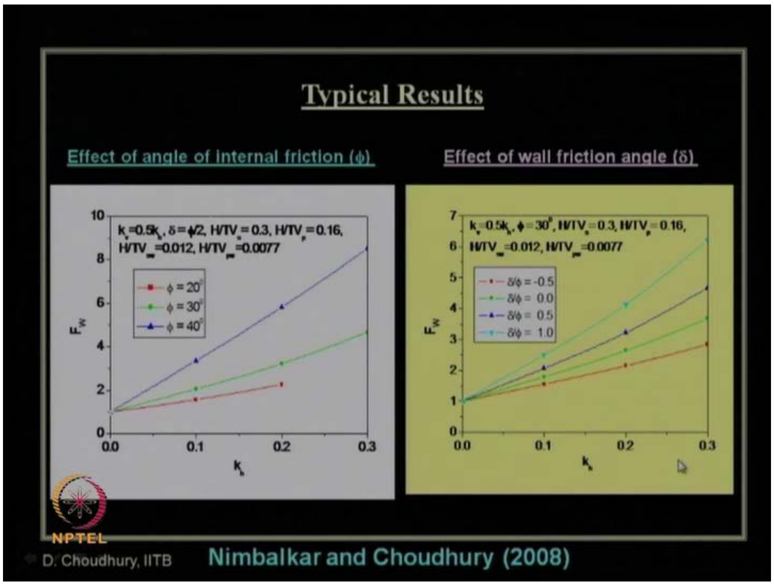

Let us look at some typical results of how the effect of soil friction angle on this F w will occur and how the wall friction angle effect on this F w also will occur.

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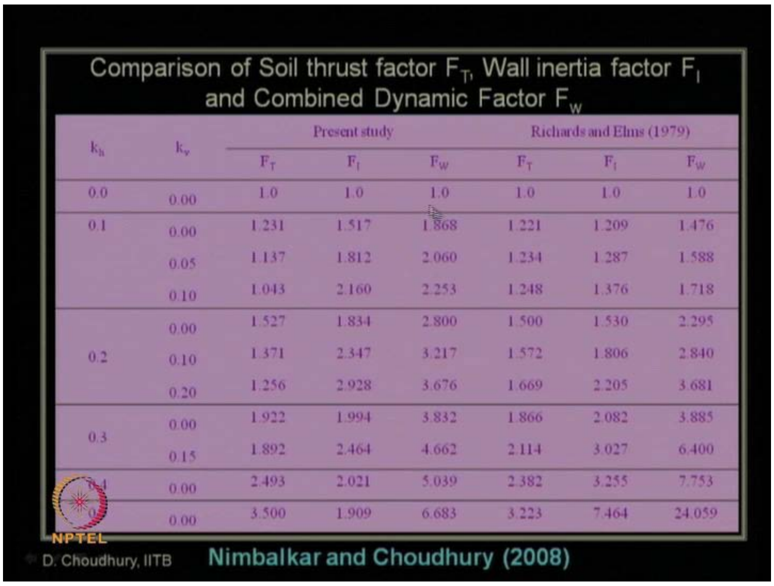

Comparison of Soil thrust factor F_T, Wall inertia factor F_I and Combined Dynamic Factor F_W

k_h	k_v	Present study			Richards and Elms (1979)		
		F_T	F_I	F_W	F_T	F_I	F_W
0.0	0.00	1.0	1.0	1.0	1.0	1.0	1.0
0.1	0.00	1.231	1.517	1.868	1.221	1.209	1.476
	0.05	1.137	1.812	2.060	1.234	1.287	1.588
	0.10	1.043	2.160	2.253	1.248	1.376	1.718
0.2	0.00	1.527	1.834	2.800	1.500	1.530	2.295
	0.10	1.371	2.347	3.217	1.572	1.806	2.840
	0.20	1.256	2.928	3.676	1.669	2.205	3.681
0.3	0.00	1.922	1.994	3.832	1.866	2.082	3.885
	0.15	1.892	2.464	4.662	2.114	3.027	6.400
0.4	0.00	2.493	2.021	5.039	2.382	3.255	7.753
[illegible]	0.00	3.500	1.909	6.683	3.223	7.464	24.059

Nimbalkar and Choudhury (2008)

D. Choudhury, IITB

These are some comparison of the proposed dynamic F w factor, with respect to that given by Richard and Elms in 1979; because Richard and Elms also proposed the similar factor. Only thing, their combined dynamic was based pseudo static analysis and in our study in this present study of “Nimbalkar and Choudhury2008”.ThisF w factor is obtained using pseudo dynamic approach.

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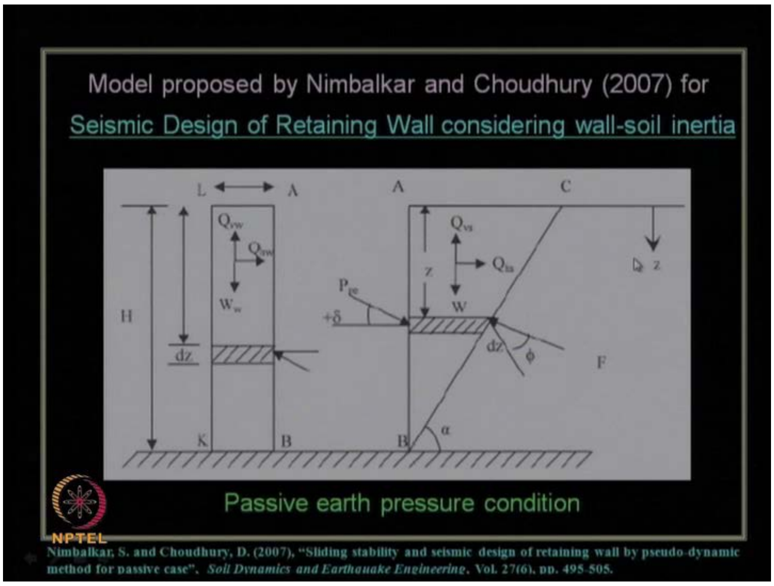

So, these are the comparison of the results as you can see over here. The similar exercise was carried out for the seismic design of retaining wall under the passive state of earth pressure. So, for passive state of earth pressure, this is the publication; "Nimbalkar and Choudhury2007" in the journal "Soil dynamics and Earthquake engineering". This is the volume and issue and these are the page numbers.

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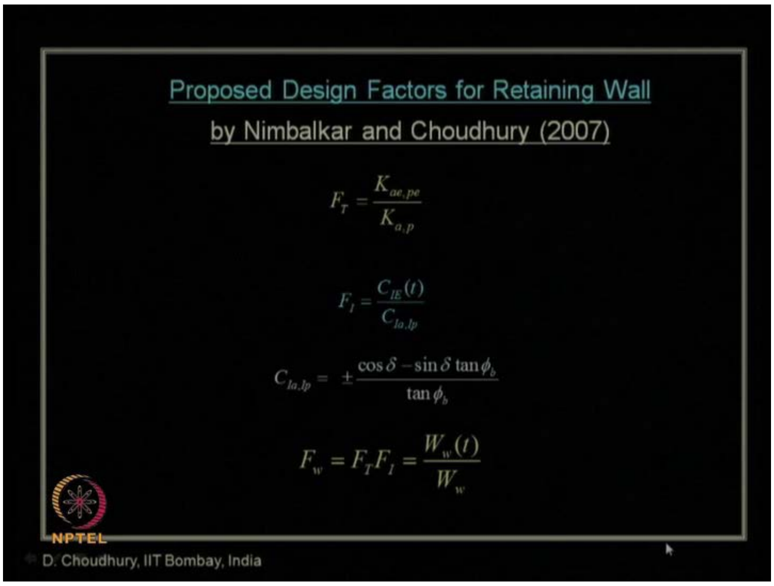

Here also the soil thrust factor, wall inertia factor and combined dynamic factor are proposed in the similar fashion.

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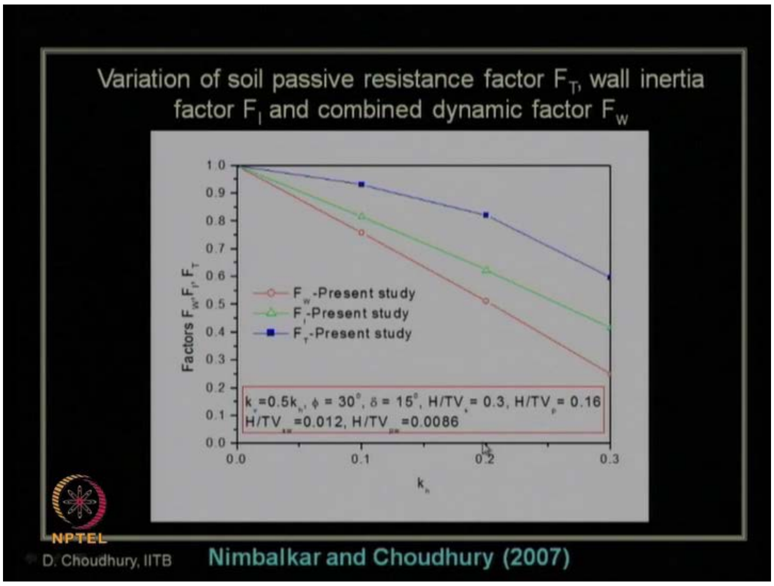

But in this case, as you know the seismicity increasing means, you require a lower factor because the passive case is always on the receiving end. It is a resistance provided by the soil. So, you are getting… a lighter weight of the wall should be required for withstanding the seismic passive earth pressure. That is why many a times for the wall design we exclude the passive state, unless it is clearly known. Mostly, we design the wall for the active state of earth pressure. Passive state; we use mostly for the bearing capacity, anchor uplift capacity, all those things as I have already discussed in the previous lecture.

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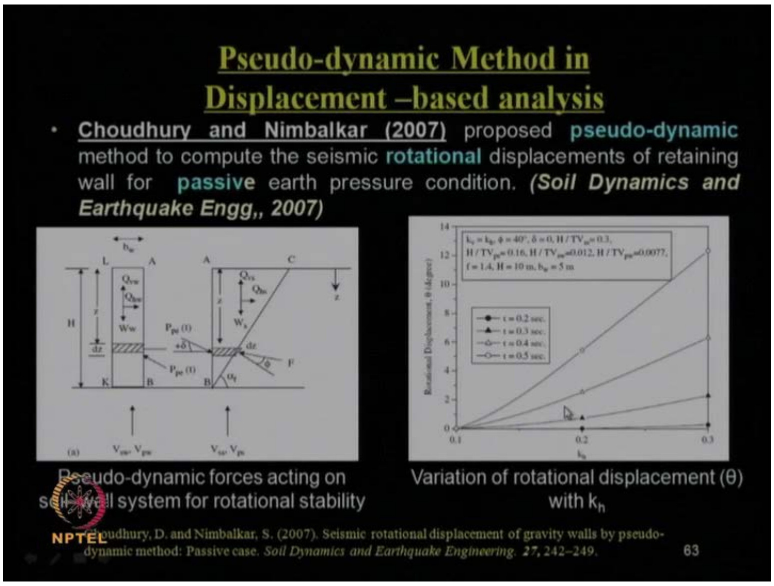

Then, also we had proposed the displacement based analysis, what has been mentioned just now in terms of the sliding mode. In terms of rotational displacement also, we should check the pseudo dynamic method application. In case of passive earth pressure condition, “Choudhury and Nimbalkar2007”; it is another journal paper in the “Soil dynamics and Earthquake engineering” of this issue of June issue of volume 27, where not the sliding of the wall is considered. Earlier, we consider the sliding mode of wall movement, now we are considering the rotational movement. That is, the moment due to this seismicity of the wall and soil, how it creates the rotational movement and how much rotational displacement one can allow for a wall to design. That permissible rotational, based on that one can easily design this wall based on these design charts of rotational displacement versus this seismicity coefficients.

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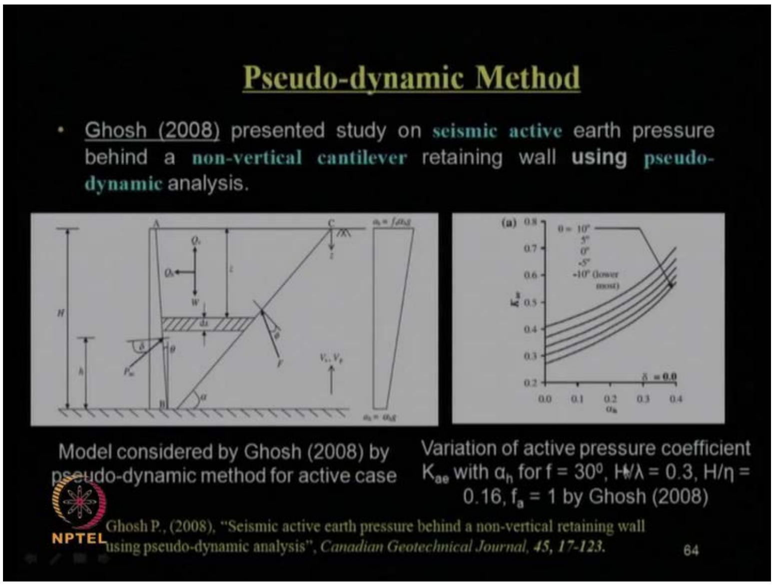

Other researcher also nowadays started using extensively this proposed pseudo dynamic model like Ghosh in 2008 proposed an extended this seismic active earth pressure analysis form a vertical wall. What we have done? a vertical wall to a non-vertical wall. So, Ghosh proposed the analysis of seismic active earth pressure, determination for a non-vertical wall using the same pseudo dynamic approach what we had proposed. So, this is the paper "Ghosh2008", which is available in "Canadian Geotechnical Journal" of issue 45.These are the page numbers. You can see this is the way we had considered the soil amplification. As I have already mentioned, linear variation. And these are the values of Ka e design charts based on different wall inclination. You can see, we had proposed for wall inclination of 0.This theta, it is with respect to vertical as you can see from this picture. So, as wall inclination changes the value of Ka e also changes as proposed by Ghosh in 2008.

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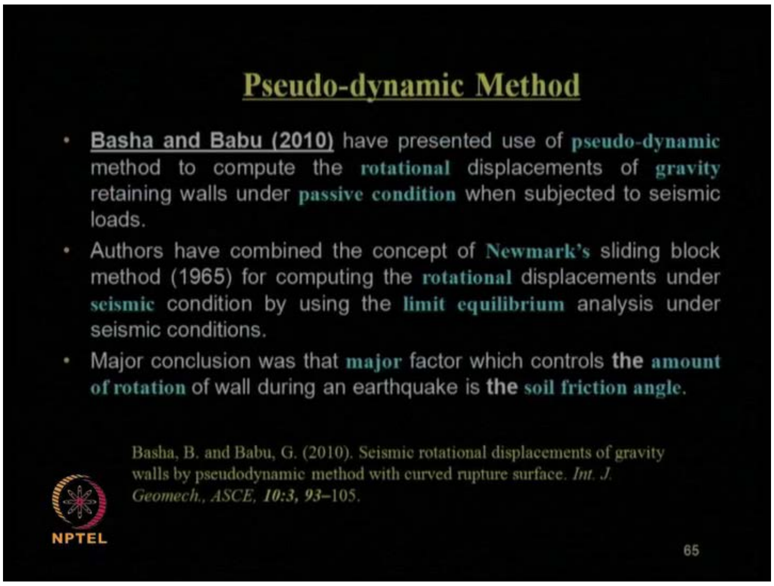

Other researchers like Basha and Babu in 2010, they also used this same pseudo dynamic method proposed by us for computation of rotational displacement for gravity retaining wall under passive state of earth pressure. They used the Newmark’s sliding block method to obtain the rotational displacement. I will describe the Newmark’s sliding block method later on, when we talk about the slope stability aspect. So, these are the details one can easily go through this paper “Basha and Babu in 2010” International journal of Geomechanics of ASCE volume 10.These are the page numbers.

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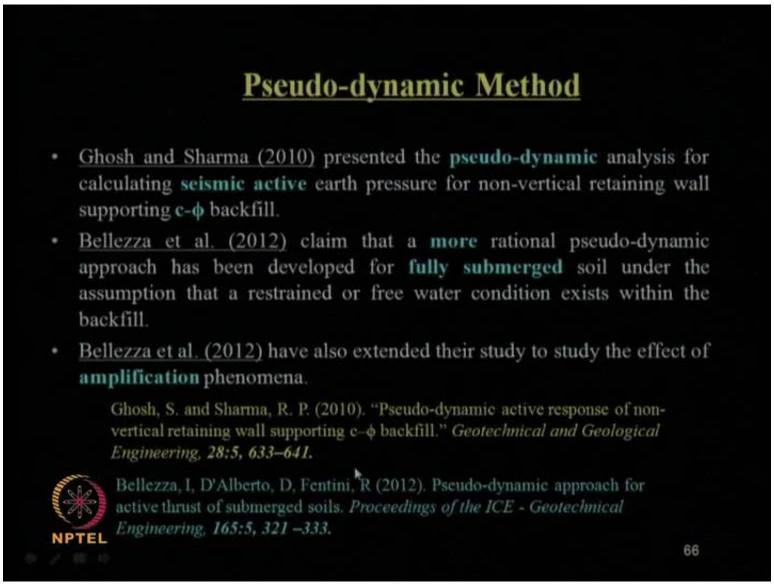

Other recent researcher like Ghosh and Sharma in 2010, they also use the pseudo dynamic model for seismic active earth pressure. For a non-vertical retaining wall, what was the addition they made? They used the C phi backfill. That is, first initially we had proposed it for only cohesion less backfill. Then Ghosh2008 used it for non-vertical wall and Ghosh and Sharma extended it further for C phi soil. And the paper is available in this “Geotechnical and Geological Engineering”, “Springer publication”; this volume, this issue, this page number.

And Bellezza et al in 2010, they extended this pseudo dynamic approach further to consider the fully submerged condition of the water to take care of the effect of amplification also, which is available in this journal paper of proceedings of the ICE Geotechnical Engineering. This is the volume number. I will discuss about the effect of water table and effect of water on the pseudo dynamic model or the earth pressure estimation on the wall very soon, subsequently in some of the slides; when we will talk about the design of retaining wall in the water front or close to sea.

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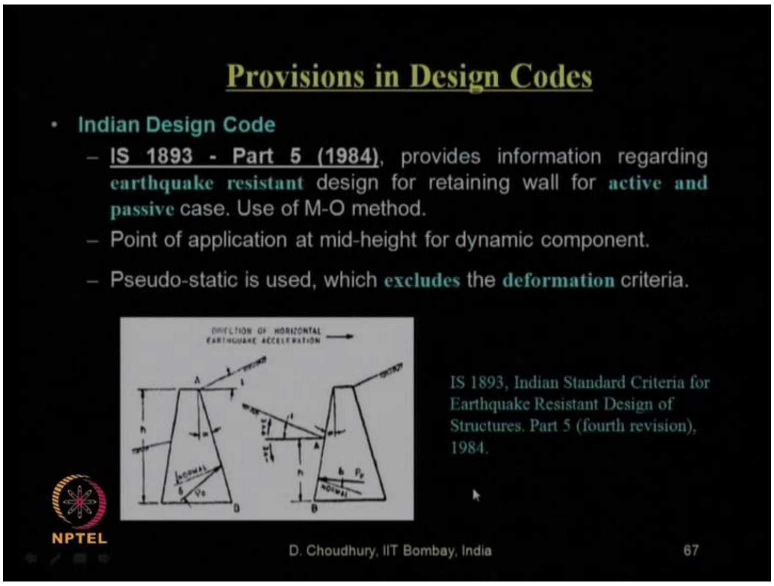

Now let us understand, what various countries design code proposes, how the retaining wall should be designed under the earthquake condition. So, first let us start with our Indian design code. As I have mentioned Indian design code, the latest version of 2002 is available only for part one; which describes about the seismic zonation and the basic soil property, spectral acceleration, etcetera. But the other parts are the latest version is still of the latest1984 version IS: 1893 part 5, which talks about the design of this retaining wall under active and passive state. It states that, use the Mononobe and Okabe method.

So, point of application is considered to be at mid height of the dynamic component. That means, if the static component acts at one-third from the base, they mentioned the extra component of the earth pressure, which is due to the dynamic case or the seismicity case that extra component consider at the mid height, instead of considering what we have found above one-third based on the combined analysis. So, the IS code suggests consider the, it is acting at the mid height only for the dynamic component and for the static component at the one-third. Then you can use the two values to find out the, where the net earth pressure is acting. So, pseudo static method is used and it excludes the deformation criteria. There is no displacement based criteria, which is mentioned in this Codal provision.

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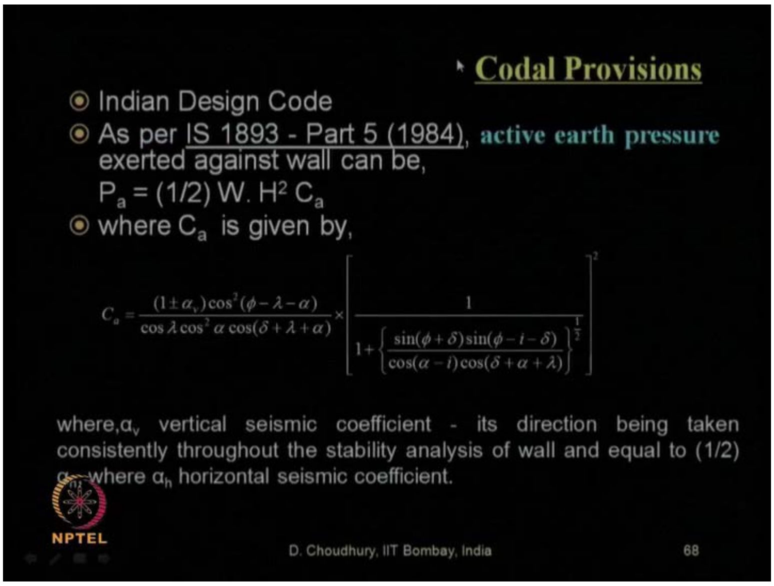

And, what are the steps? This is the formula needs to be used Pa equals to half W H square C a; where C a is given by this expression. This is nothing but seismic active earth pressure coefficient as proposed by Mononobe-Okabe. So, this is basically propose the Mononobe-Okabe needs to be used and it says that alpha v is the vertical acceleration coefficient, which should be taken as half of that alpha H; that is, horizontal seismic acceleration coefficient. So, this is with respect to the passive case.

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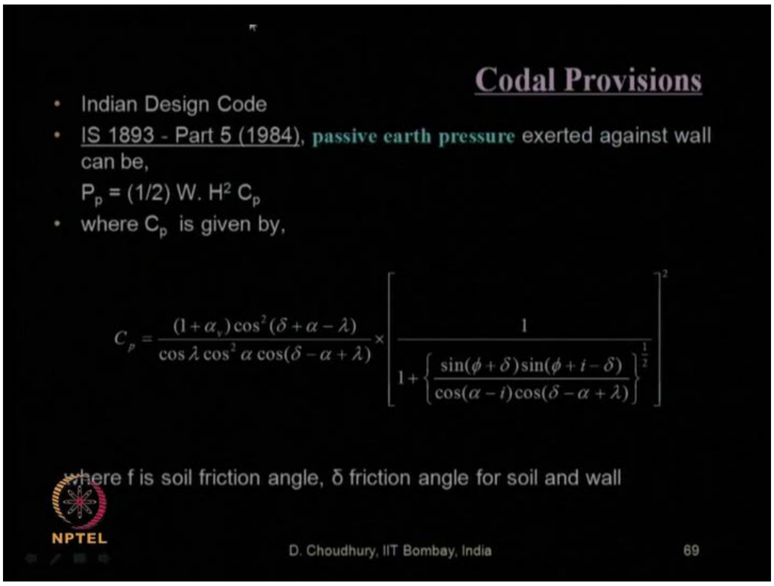

Passive case, similar expression has been given by the code. And the expression for that passive earth pressure coefficient is nothing but the Mononobe-Okabe's passive earth pressure coefficient.

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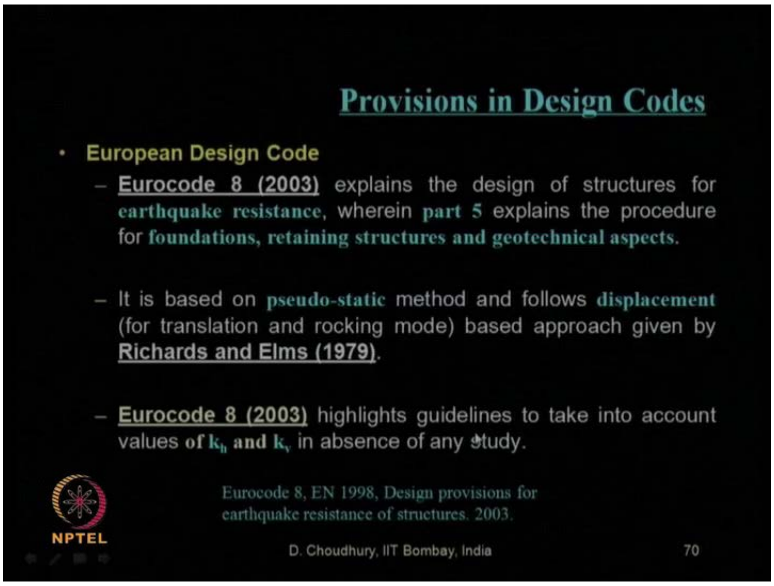

Now coming to European code, European design code, Eurocode 8 of 2003 version. It suggest the design of retaining structure, it is based on the "Richard and Elms" displacement based criteria. Though it is pseudo static, but it considered the displacement based criteria also. And there is a guideline to consider this k h and k v values in absence of a particular study at the soil site.

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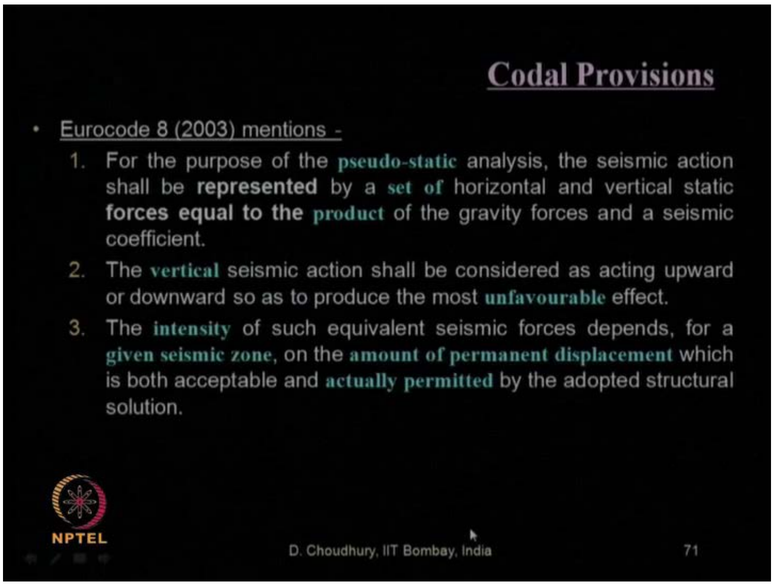

So, this is the details about the Eurocode. You can see over here, it says again the pseudo static method. As I said, a set of horizontal and vertical forces should be considered. Vertical seismic acceleration should be considered as acting upward or downward, whichever gives the most unfavorable effect. That is, most critical condition. As we have also discussed. And the intensity of such equivalent force for a given seismic zone on the amount of permanent displacement should be acceptable.

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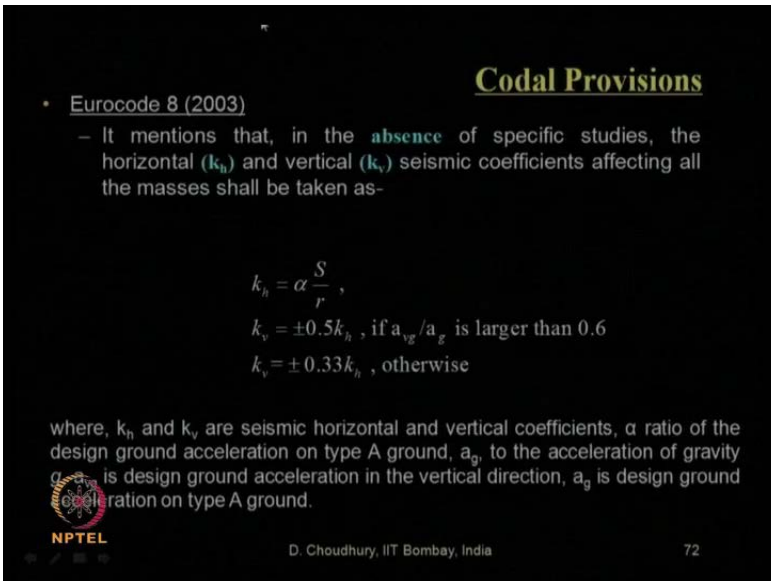

So based on that, it suggests these are the steps how to calculate in absence of specific values when you do not have the site specific ground response analysis done in that case or the seismic hazard analysis, if you have not done for your site. In absence of those cases, you can estimate the value of k h and k v for your design using this pseudo static approach as per Eurocode 8 in this manner. That is, k h is alpha times S by r. These are some coefficients which can be obtained from the tables given in this Eurocode 8. k v should be considered as half of this k h plus or minus because of depending on which direction gives the most critical value and this half should be considered when this a v by a g is larger than 0.6.And, otherwise it should be considered one-third of k h as I was telling earlier.

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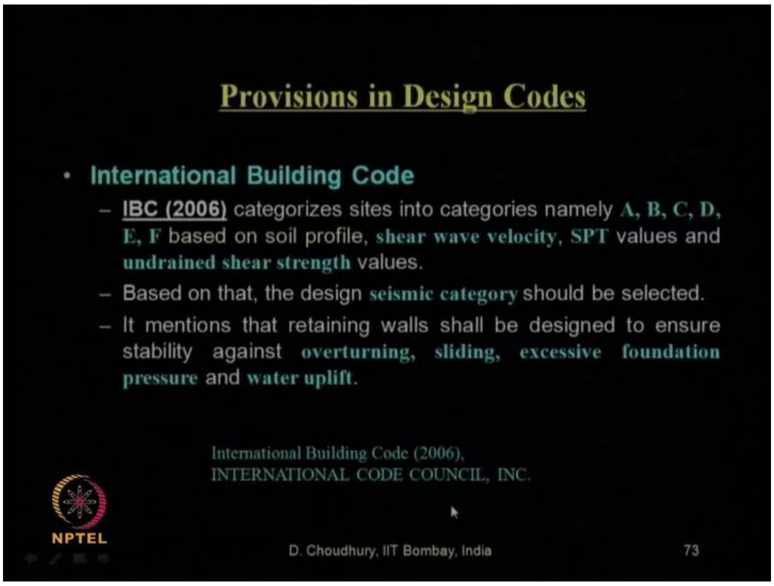

Whereas International Building Code, which we call as IBC 2006 is considering the various sites, as I have already described, there are six different types of site as proposed by IBC in similar line of NEHRP guideline like A, B, C, D, E site; based on their shear wave velocity and SPT value, undrained shear strength value, all these are input values. Based on that, the overturning, sliding, excessive foundation pressure and water uplift, all these cases needs to be checked.

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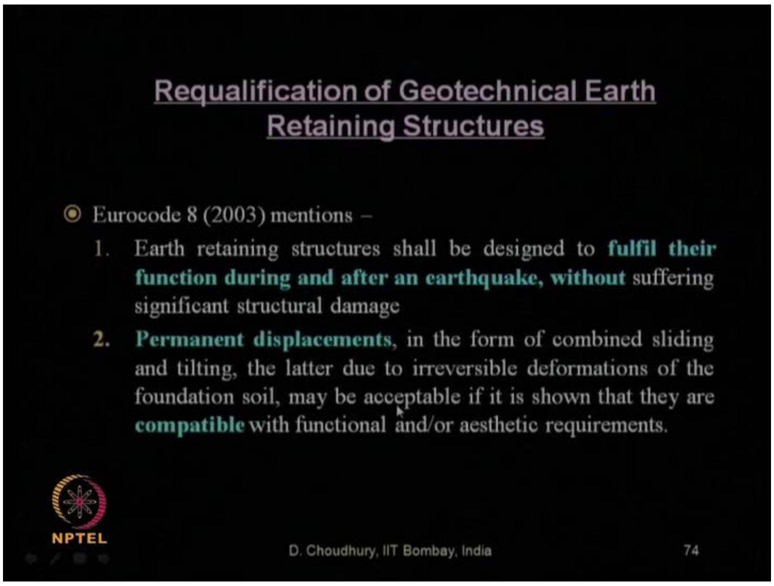

And for that, what is proposed? The Eurocode mentions, so if we do the requalification of geotechnical earth retaining structures under this seismic condition like Eurocode suggests that, retaining factors should be designed to fulfill their function during and after an earthquake without suffering from significant structural damage. That is the basic guideline. And this permanent displacement, which you can easily compute using Richard and Elms method or that is, if you use pseudo static method or our proposed method of Choudhury and Nimbalkar for sliding and rotational stability using pseudo dynamic approach in the form of combined sliding and tilting. And the latter due to irreversible deformations of the foundation soil may be acceptable, if it is shown that they are compatible with the functional under aesthetic requirements.

That is, you have to also check not only the permissible displacement or permanent displacement should be within the permissible value, but it should be within the aesthetic requirement. That is, if you do not allow your wall which is inclined like this. It can be; if you look at here, if your wall is inclined like this, it can be also a safe if it is retaining wall back fill in this direction. In this direction also, it can be designed as a safe section. Everything can be designed, but aesthetically you will never like a wall which is inclined like this and tilting on the backfill side in this manner, right. So, that is why it says not only the permissible displacement as per this displacement criteria, but also the aesthetic requirement also needs to be checked.

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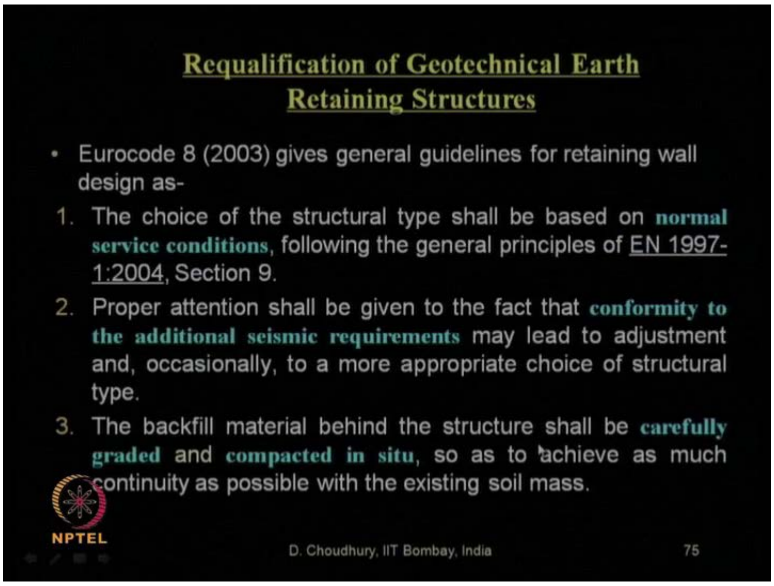

Then, what are the other guidelines? Like the choice of structural type, based on the normal serviceability conditions has to be followed. That is, under the static condition. Everything should be satisfied, then you have to check these dynamic conditions. That additional seismic requirements needs to be checked and carefully graded. The backfill material should carefully graded and compacted that in situ so that, it achieve as much continuity as possible with existing soil mass, whatever value it has been used for the design.

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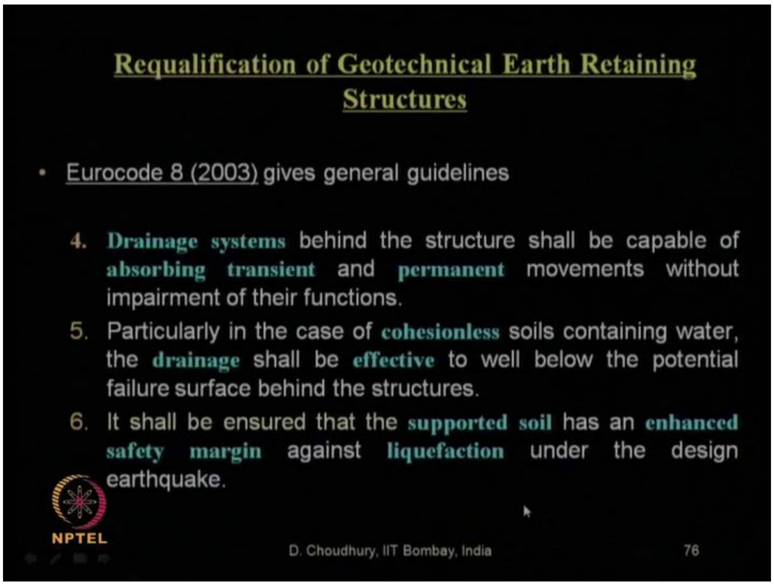

Like drainage system behind the structure, should always be present. Because if you provide the drainage there will be no extra water pressure coming on the wall. As we all know in the static case of wall design, the same thing also acts here for the dynamic case as well.

But there are few cases, where you cannot avoid the drainage like, if it is a water front wall or sea wall. That is why I will discuss that separately. Now, particularly in case of cohesion less soil containing water, the drainage shall be effective to well below the potential failure surface behind the structure. So, you have to drain out it below its potential failure surface so that, it does not fail within that region. And it shall be ensured that the supported soil has an enhance safety margin against the liquefaction. That means, the backfill soil which you are using the must not liquefied; because if it is liquefying, then automatically you are getting nothing but water standing behind your wall, not the earth which is supposed to be standing there.

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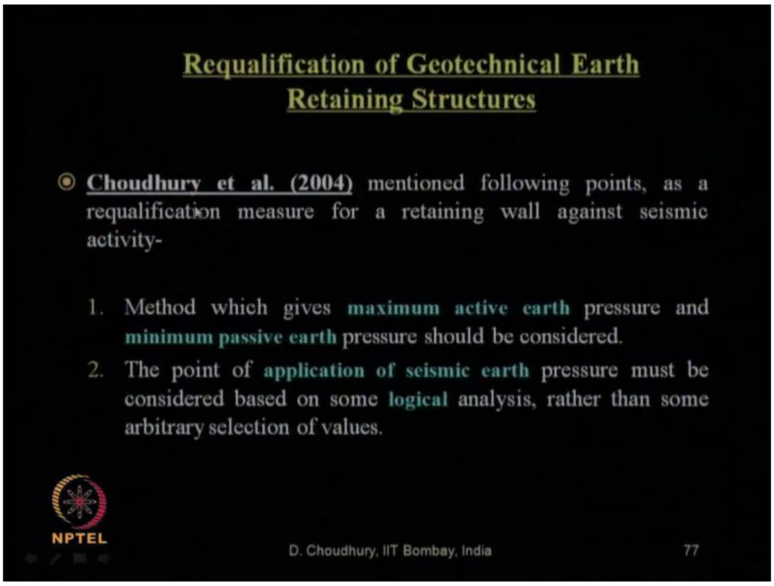

Also other guidelines which was proposed by our paper "Choudhury et al. 2004" of current Science, as I have already given this detail. It says that, method which gives maximum active earth pressure and minimum earth, passive earth pressure that should considered for any design which is obvious for a safe design for retaining wall. And the point of application of the seismic earth pressure must be considered based on some logical analysis. Logical analysis means, what we have shown the steps of analysis using horizontal slice method, etcetera, rather than some arbitrary selection of values; because by that manner, what we can do? Sometimes we when we can provide the economy in the design. So, these ensures the safety and these ensures the economy of your design. Why unnecessarily you should put a more reinforcement in your RCC rigid wall, when you know actual point of application of your seismic earth pressure.

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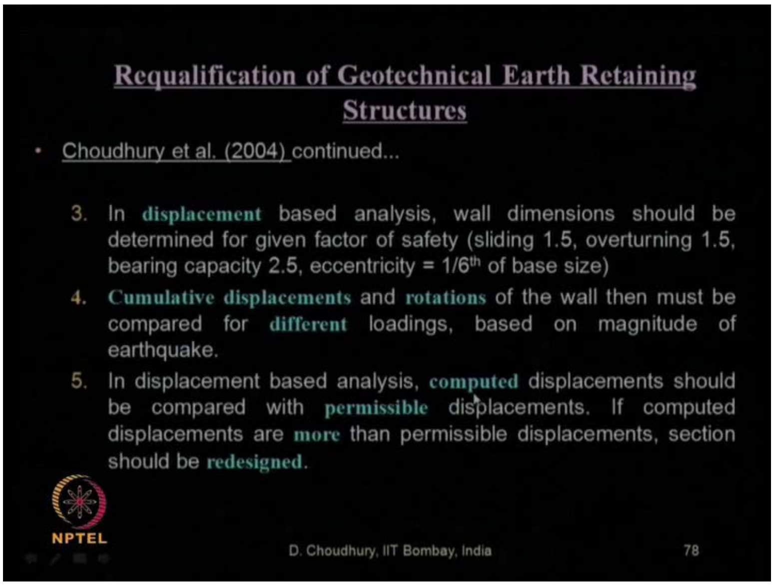

Also the displacement based analysis mentions that, given factor of safety in static case has to be maintained in dynamic case as well. Like for sliding 1.5, overturning 1.5, bearing capacity 2.5 and eccentricity should be one-sixth of the base size. That means there should be no tension right. So, that has to be maintained and cumulative displacement and rotations of the wall, then must be compared with the different loading conditions based on the magnitude of the earthquake. And that should be within the permissible range what is given or what is designed or what is proposed in the given document or at a particular site for particular design.

And in the displacement based analysis, computed displacement should be compared with the permissible displacement. If the computed displacements are more than the permissible displacement, then what need to be done? Section should be redesigned. That is, another section as I have already proposed, like those combined dynamic design factor F w should be rechecked and you have to do an iterative design process until you get the safety in terms of the factor of safety, in terms of sliding mode of failure, overturning mode of failure, bearing capacity failure and eccentricity as well as the displacement criteria. That is, displacement also will be in the safer range. Fine.

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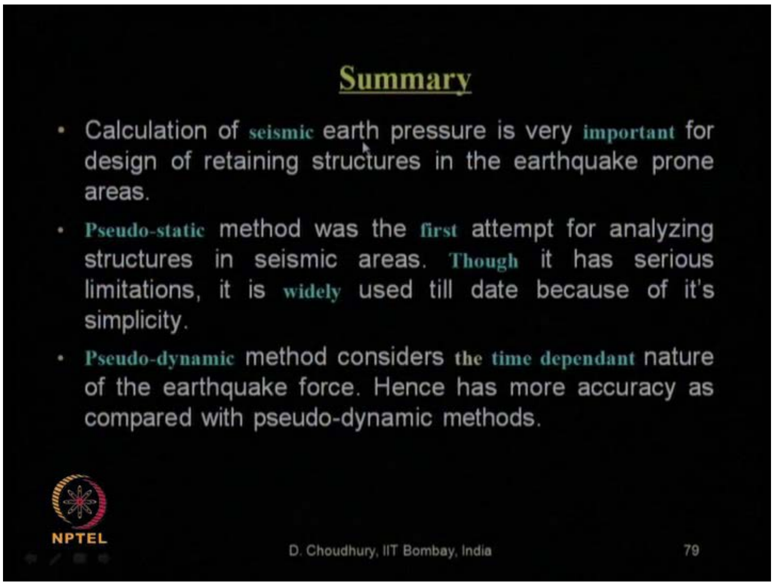

So, with this we can summarize that calculation of the seismic earth pressure is very important for design of retaining structures in the earthquake prone areas. And pseudo static method was the first attempt for analyzing the structure in seismic areas, though it has a serious limitation; because it does not consider any dynamic nature of the problem of Earthquake Engineering; whereas, pseudo dynamic method considers the time dependency of the nature of the earthquake force, also soil amplification. Hence, it has more accuracy compared to the pseudo static method.

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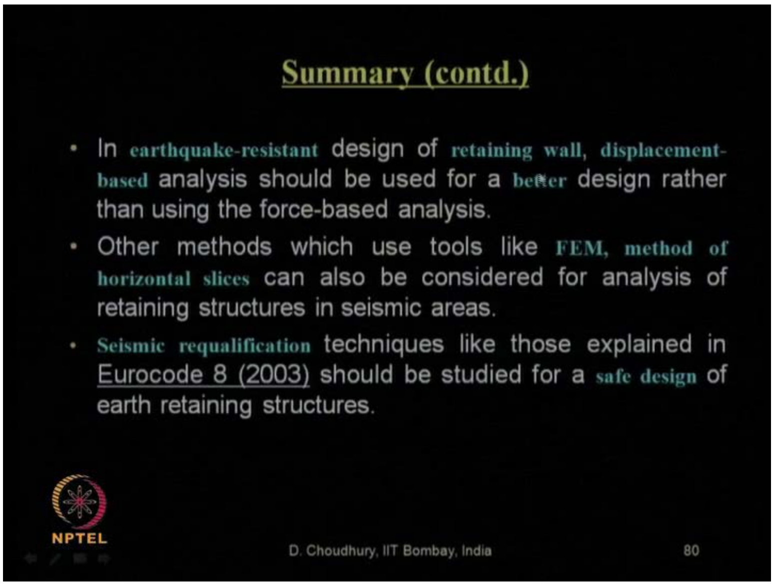

And, in earthquake resistant design of this retaining wall, displacement based analysis should be used; because it provides a better design guideline, rather than force based analysis. Because in force based, you only consider the total force of for the design, but not the point of application as well as its displacement of the wall, which is more critical.

So, displacement based approach which is related to the performance of the wall or performance based design of the wall should be considered. Other methods, which use the tool like finite element method, method of horizontal slices, etcetera, also can be considered for the analysis of retaining wall in seismic areas. And for seismic requalification technique, that is once retaining wall suppose it has experienced or failed during an earthquake, if you want to redesign it or re-qualify that retaining wall, further you need to use the proposed guideline as far as the latest code worldwide available. So, among those it has been proposed over here like, as we have discussed here Eurocode 8 gives a fairly good estimate; because it also considers another important point. It also considers the effect of soil amplification, which other codes do not consider.

So, Eurocode mentioned for different types of soil as we have already mentioned 1.4, 1.6, 1.2 amplification factors, etcetera, when we have shown the Eurocode. So, that can be considered for a safe design of earth retaining wall depending on your site condition, depending on your soil condition, depending on your earthquake input motion, etcetera. So, with this we have come to the end of today's lecture, we will continue further in our next lecture.