

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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Pseudo-static analysis

$$F_h = \frac{a_h W}{g} = k_h W$$

As per Terzaghi (1950)

$$F_v = \frac{a_v W}{g} = k_v W$$

As per Gutenberg and Richter (1956), $a_0 = k_h g$ and computed as,

$$\log_{10} a_0 = -2.1 + 0.81M - 0.027M^2$$

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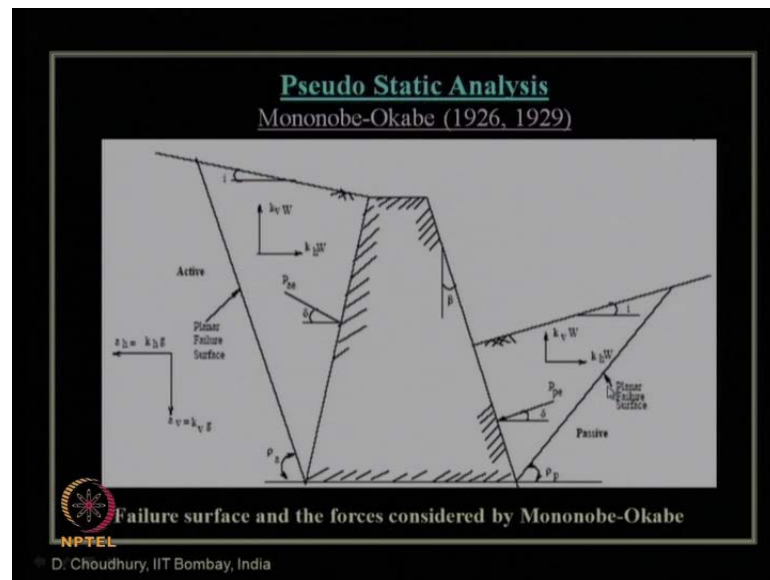
So, a quick recap what we have learnt in our previous lecture. We have mentioned about mostly force based analysis and displacement based analysis. And within force based analysis we mentioned pseudo static analysis is the basic method by which the analysis for the seismic earth pressure acting on the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

$$P_{ac,pe} = \frac{1}{2} \gamma H^2 (1-k_v) K_{ac,pe}$$

$$K_{ac,pe} = \frac{\cos^2 (\phi \mp \beta - \theta)}{\cos \theta \cos^2 \beta \cos (\delta \pm \beta + \theta)} \left[1 - \left(\frac{\sin (\phi + \delta) \sin (\phi \mp i - \theta)}{\cos (\delta \pm \beta + \theta) \cos (i - \beta)} \right)^{0.5} \right]^2$$

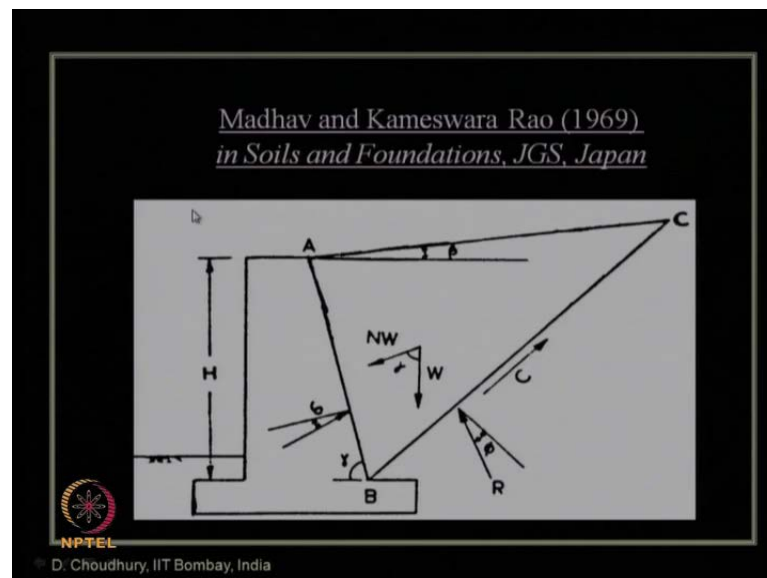
$$\theta = \tan^{-1} \left[\frac{k_h}{1 - k_v} \right]$$

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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_{pe} is nothing but seismic passive earth pressure and P_{ae} is nothing but seismic active earth pressure. Total value of the pressure is $\frac{1}{2} \gamma H^2 \times (1 - k_v) \times K_{ae}$ and or K_{pe} . That is, for active case it is K_{ae} ; that is, coefficient of active earth pressure coefficient on the seismic condition. And K_{pe} is seismic passive earth pressure coefficient, k_v is the vertical seismic acceleration coefficient, H is the height of the wall, γ is the unit weight of the backfill material.

So, the expressions of K_{ae} or K_{pe} 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_{pe} , one should use the lower symbol; that is here plus, here minus, here minus and here plus; where in this expression, θ is nothing but $\tan^{-1} \frac{k_h}{1 - k_v}$. And other parameters like, δ is wall friction angle and ϕ is the soil friction angle and this β is nothing but the inclination of the wall, which is called wall batter and I is nothing but inclination of the ground.

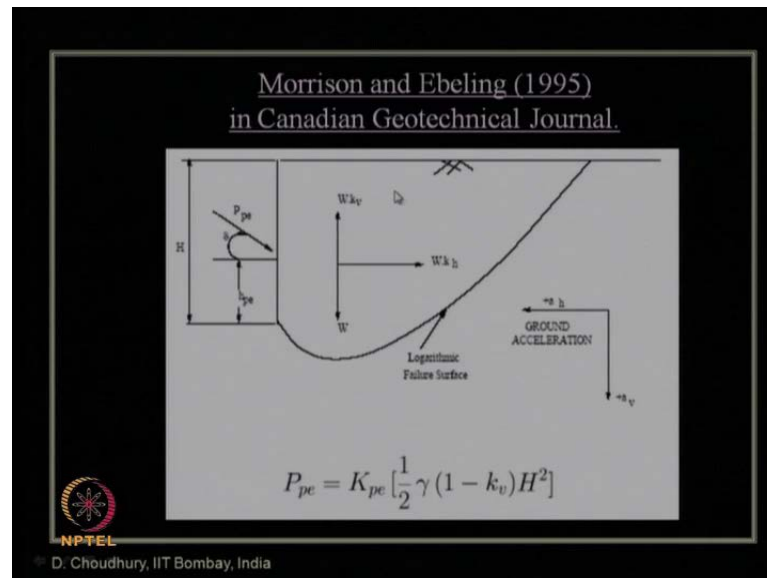
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Other researchers like Madhav and Kameswara Rao in 1969 in their journal publication in soils and foundations published by Japanese Geotechnical Society in Japan. They also estimated the seismic active and passive earth pressure using this pseudo static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

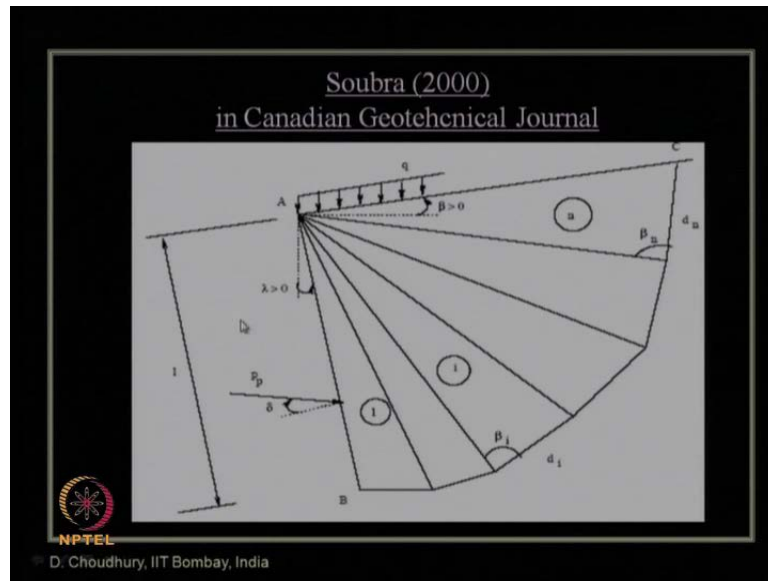
But they have used C phi type of soil that is the generalized soil, whereas Mononobe Okabe approach was only for the cohesion less soil.

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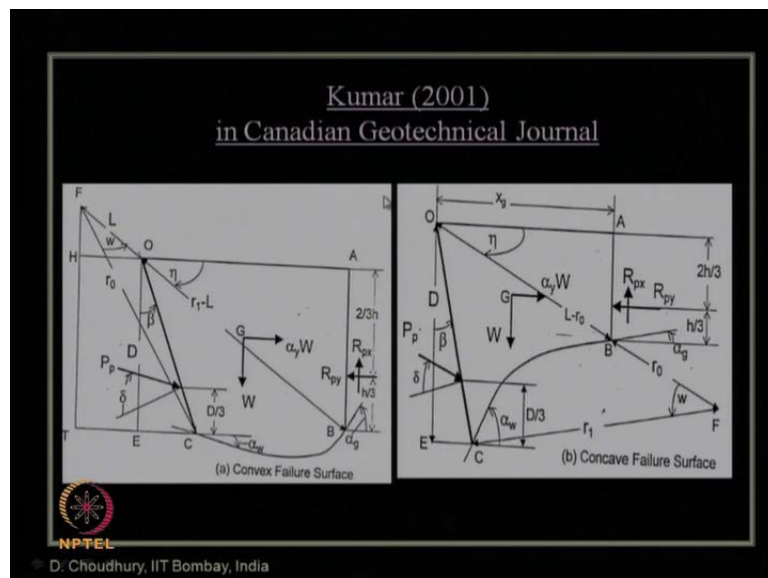
Then Morrison and Ebeling in 1995 in their journal paper “Canadian Geotechnical journal”, they proposed for a rigid retaining wall how to estimate the passive earth pressure considering curved linear. That is instead of a planner failure surface, curvilinear failure surface and the shape of that curvilinear 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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Seismic Passive Earth Pressure / Resistance

Case 1: Positive delta case

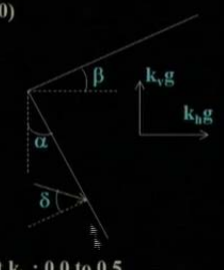
Seismic passive force P_{pd} is divided into three components as,


- (i) Unit weight component $P_{p\gamma d}$ ($\gamma \neq 0, q = c = 0$)
- (ii) Surcharge component P_{pqd} ($q \neq 0, \gamma = c = 0$)
- (iii) Cohesion component P_{pcd} ($c \neq 0, \gamma = q = 0$)

$$P_{pd} = P_{p\gamma d} + P_{pqd} + P_{pcd}$$

Variation of parameters

1. Wall batter α : $-30^\circ \leq \alpha \leq 30^\circ$
2. Ground inclination β : $-30^\circ \leq \beta \leq 30^\circ$
3. Soil friction angle ϕ : 10° to 50°
4. Wall friction angle δ : 0 to ϕ
5. Wall adhesion c_s : 0.0 to $(\tan \delta / \tan \phi)c$
6. Horizontal seismic acceleration coefficient k_h : 0.0 to 0.5
7. Vertical seismic acceleration coefficient k_v : $0.0k_h, 0.5k_h, 1.0k_h$




 Subba Rao, K. S. and Choudhury, D. (2005), "Seismic passive earth pressures in soils", *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, USA, 131(1): pp. 131-135.

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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Composite Failure surface and forces considered
[Subba Rao and Choudhury, 2005]

$$= \frac{\pi}{4} - \frac{\phi}{2} + \frac{1}{2} \tan^{-1} \left(\frac{k_h}{1 - k_v} \right) + \frac{\beta}{2} - \frac{1}{2} \sin^{-1} \left(\frac{\sin \left(\tan^{-1} \left(\frac{k_h}{1 - k_v} \right) - \beta \right)}{\sin \phi} \right)$$

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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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Determination of $K_{p\gamma d}$ ($\gamma \neq 0, q = c = 0$)

$$M_{P_{p\gamma d}} = M_{W1d} + M_{P_{p\gamma R}} - M_{W2kv} - M_{W2kh}$$

Where,

- $M_{P_{p\gamma d}}$ = moment of $P_{p\gamma d}$.
- M_{W1d} = moment of the soil mass ABDGA, together with the seismic components: W_{1kh} and W_{1kv} .
- $M_{P_{p\gamma R}}$ = moment of the Rankine passive force $P_{p\gamma R}$.
- M_{W2kv} , M_{W2kh} = moments of the seismic components of the weight of portion DGE.

$$K_{p\gamma d} = \frac{2P_{p\gamma d} \cos \delta}{\gamma H^2}$$

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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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Determination of K_{pcd} ($c \neq 0, \gamma = q = 0$)

$$M_{P_{pcd}} = M_C = M_{Ca} + M_{P_{pcR}}$$

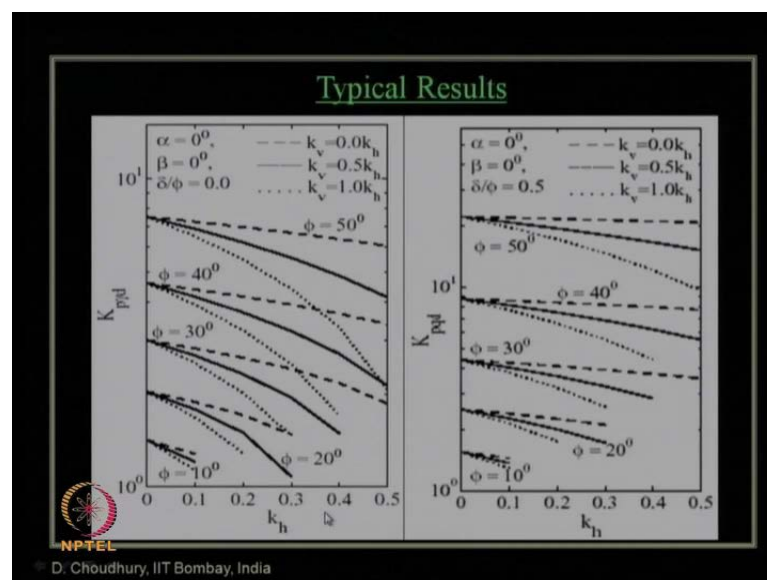
Where,
 $M_{P_{pcd}}$ = moment of P_{pcd} .
 M_C = moment of the cohesive force C (on the failure surface BD).
 M_{Ca} = moment of the adhesive force C_a (on the wall-soil interface AB).
 $M_{P_{pcR}}$ = moment of the Rankine passive force P_{pcR} .

$$K_{pcd} = \frac{P_{pcd} \cos \delta}{2cH}$$


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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

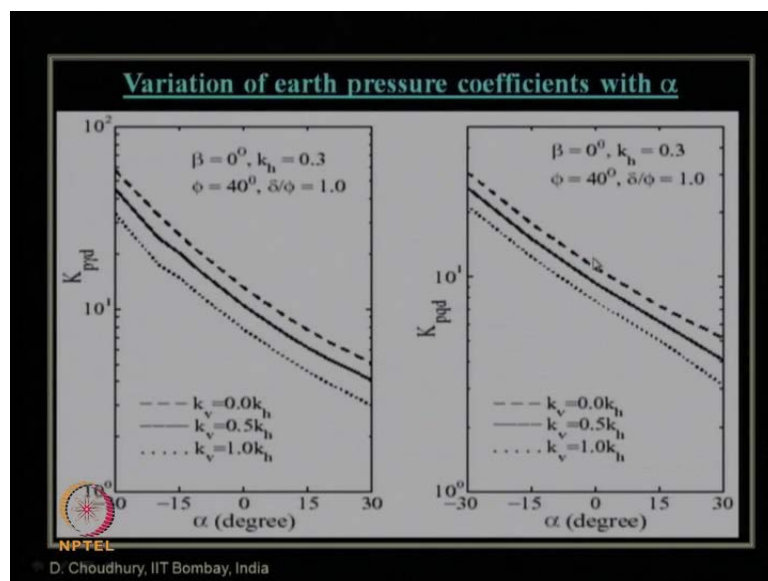
Values of K_{pzd}

Case for	ϕ (degree)	$\delta\phi$ for $c_u/c = 0.0$				$\delta\phi$ for $c_u/c = \tan\delta \tan\phi$			
		0.0	0.5	0.75	1.0	0.0	0.5	0.75	1.0
$\alpha = 0^\circ$	10	1.19	1.32	1.39	1.45	1.19	1.55	1.65	1.69
	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
$\alpha = 30^\circ$	10	0.73	0.87	0.91	0.93	0.73	0.96	1.03	1.07
	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

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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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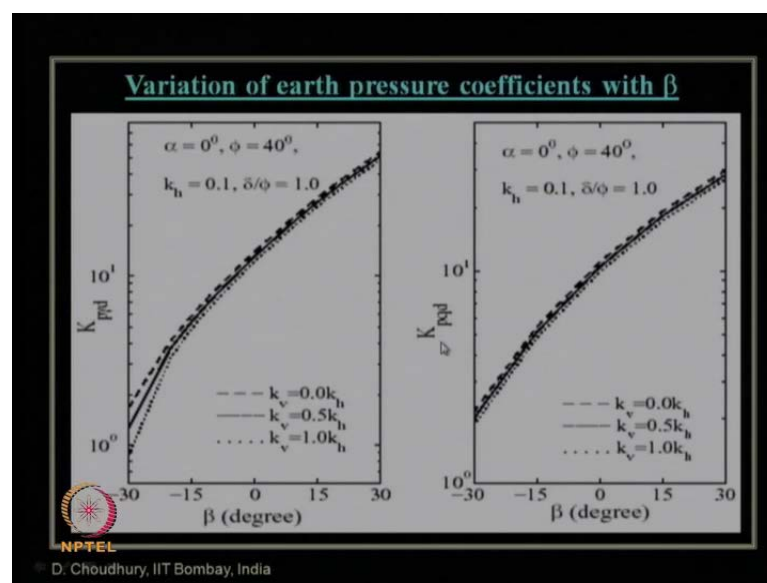


So if it is in between, then one can do a linear interpolation. Now, if one tries to get an optimum design of a retaining wall, so they should know what should be the batter angle.

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.

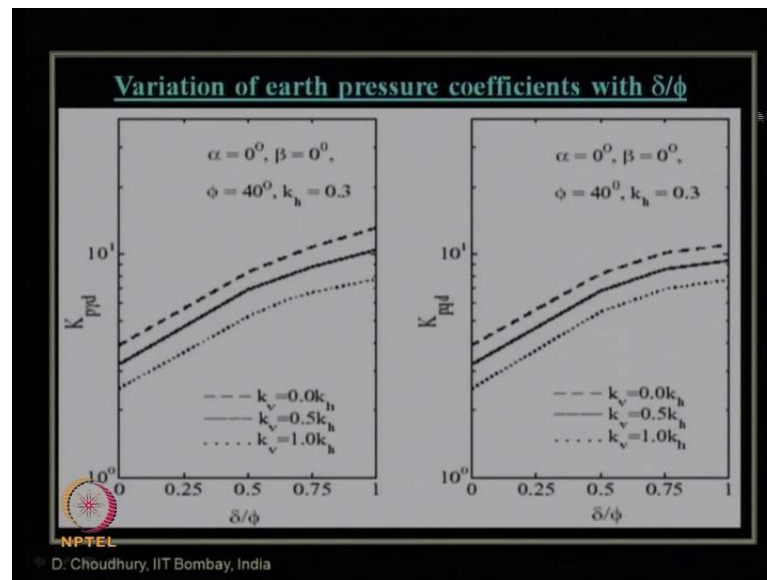
So, if we see the variation with respect to wall batter angle alpha, you can see the variation of $K_p \gamma d$. As alpha changes from negative batter to positive batter, it varies like this. Same, similar trend has been obtained for $K_p q d$.

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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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Interpolation formula

To obtain seismic passive earth pressure coefficient values for any other parameter, the interpolation formula can be used is proposed as,

$$\log (K_{p,d})_{x_i} = \log (K_{p,d})_{x_0} + \frac{\log (K_{p,d})_{x_1} - \log (K_{p,d})_{x_0}}{x_1 - x_0} (x_i - x_0)$$

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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: $\phi = 40^\circ$, $\delta\phi = 1.0$, $\gamma = 18 \text{ kN/m}^3$, $q = 15 \text{ kN/m}^2$, $c = 15 \text{ kN/m}^2$, $c_u/c = 0.0$, $\alpha = 0^\circ$, $\beta = 0^\circ$, $H = 5\text{m}$, $k_h = 0.3$)

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

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

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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 be 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 ϕ soil with surcharge, c ϕ soil without surcharge and ϕ 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 $P_p d$ and $P_p d$ star in the case of single failure surface, in $P_p d$ 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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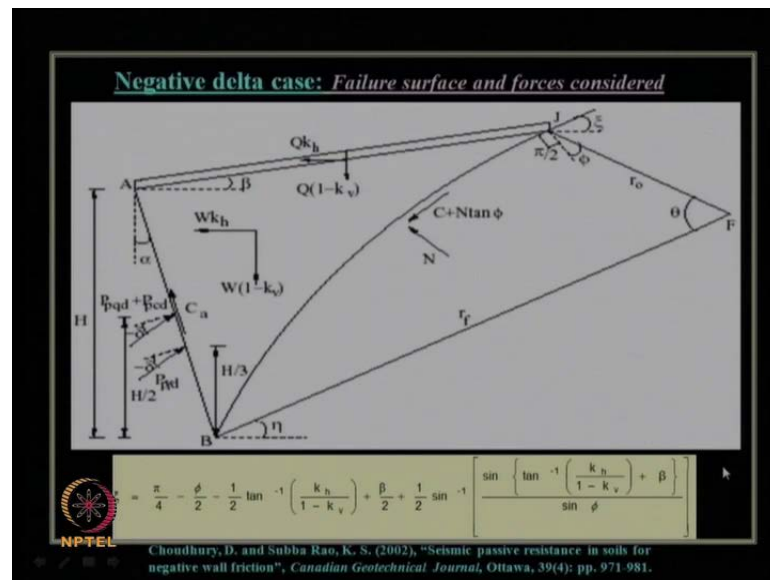
Comparison of $K_{p\gamma d}$ values obtained by present study with available theories in seismic case for $\alpha = 0^\circ$, $\beta = 0^\circ$, $\phi = 30^\circ$

δ/ϕ	k_b	k_c	Mononobe-Okabe		Morrison and Ebeling	Soubra	Kumar	Present study
			Okabe	Okabe				
0.0	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	0.0	4.360	4.198	-	-	4.099	
		0.1	4.350	4.160	-	-	3.890	
0.5	0.2	0.0	3.988	3.870	3.900	-	3.860	
		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	0.0	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	0.0	2.400	2.400	-	-	1.981	
		0.5	2.477	2.470	-	-	2.470	
1.0	0.0	0.0	8.743	6.150	5.941	5.802	5.783	
		0.1	7.812	5.733	5.500	5.500	5.400	
		0.2	6.860	5.280	5.020	5.020	5.100	
		0.3	5.875	4.940	4.500	4.500	4.750	
		0.4	4.830	4.300	-	3.900	4.100	
		0.5	3.645	3.400	-	3.200	3.300	

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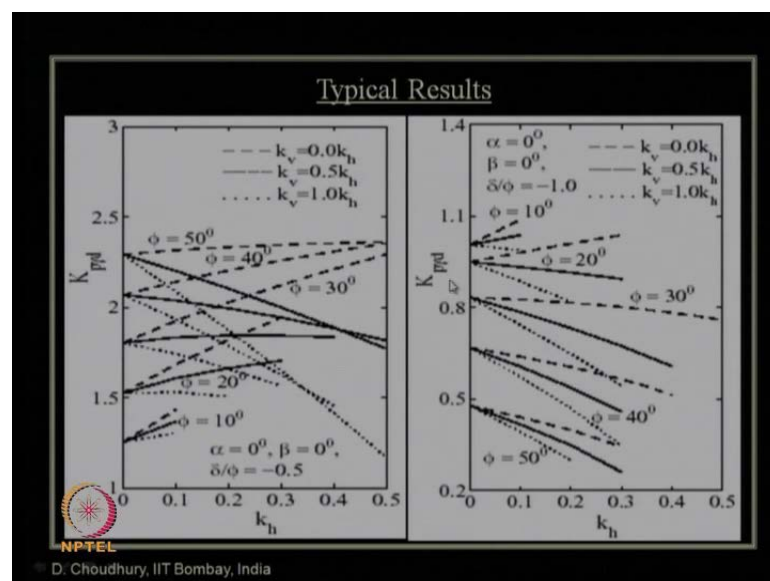
The comparison of the our proposed value of this $K_{p\gamma d}$ coefficients of seismic passive earth pressure with respect to other researchers are shown over here. Obviously, our values are far away from the Mononobe-Okabe because of the reason, Mononobe-Okabe considered the planer rupture surface 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

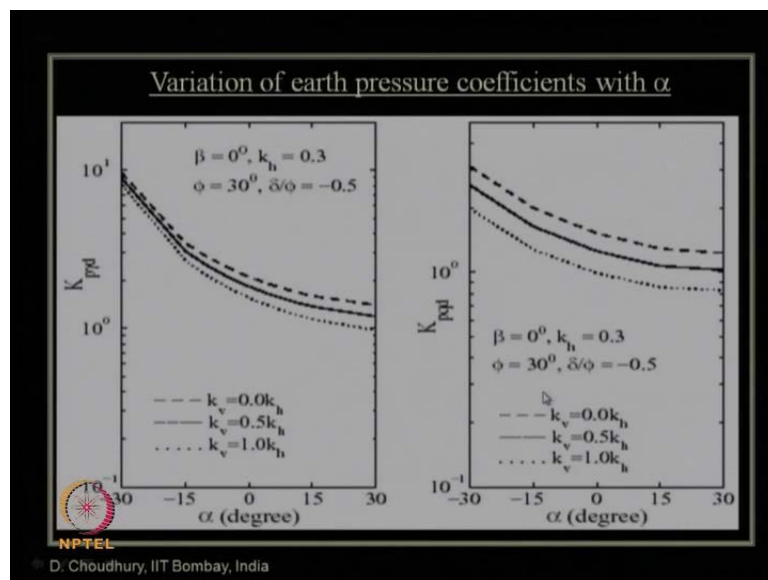
Values of K_{ped}

Case for ϕ (degree)	$\delta\phi$ for $c_u/c = 0.0$				$\delta\phi$ for $c_u/c = \tan\delta \tan\phi$			
	-0.5	-0.67	-0.75	-1.0	-0.5	-0.67	-0.75	-1.0
$\alpha = 0^\circ$								
$\beta = 0^\circ$	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	-	-	-
$\alpha = 30^\circ$								
$\beta = 0^\circ$	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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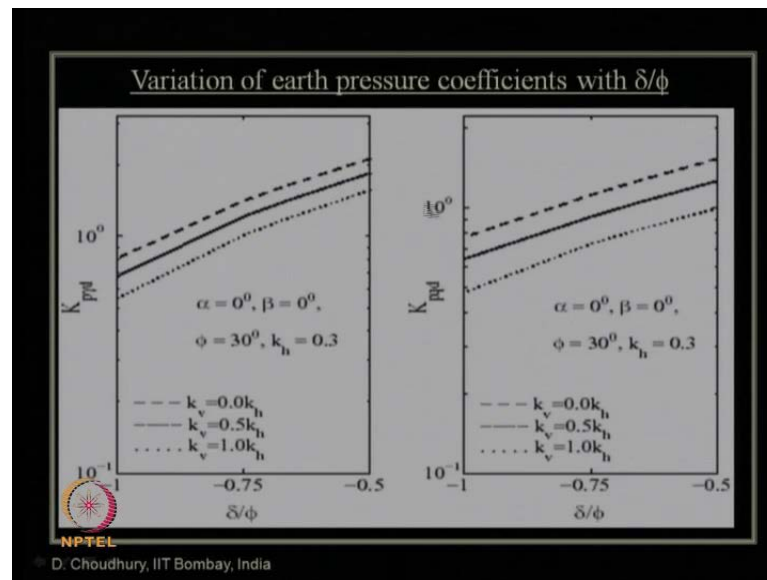
And, these are the table for $K_p c d$ for different ranges of wall adhesion. So, any wall adhesion in between, one can easily interpolate using that interpolation formula.

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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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Validation of Principle of Superposition

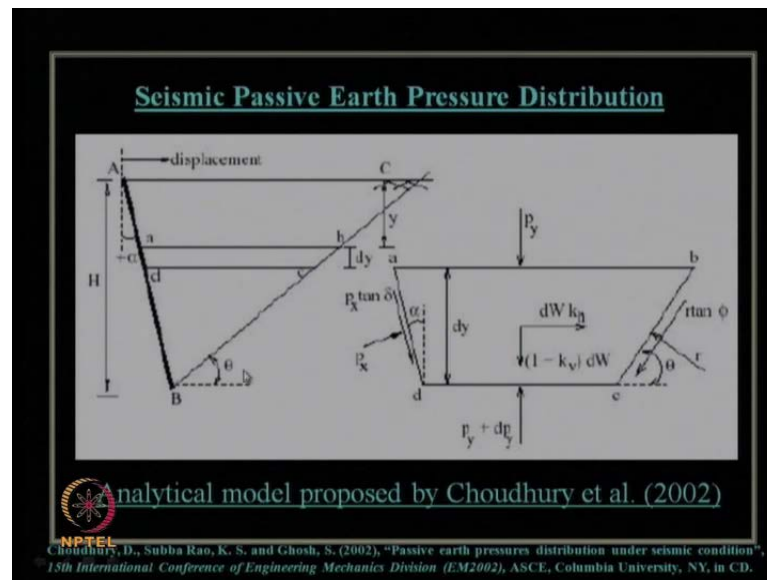
ϕ	Case	k_h, k_v	Independent critical failure surfaces				Single critical failure surface				Max.% Error ^a in K values		
			K_{ped}	η	K_{pqd}	η	K_{ped}	η	K_{pqd}	η			
30°	c- ϕ soil with q	0.0, 0.0	1.095	-14 ^o	1.764	-1 ^o	1.803	-7 ^o	1.103	1.779	1.803	-8 ^o	
		0.3, 0.3	1.095	-14 ^o	0.993	-5 ^o	1.564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

^a Error = $[(K_{pd}^i - K_{pd}^s)/K_{pd}^i] \times 100$

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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.

(Refer Slide Time: 16:05)



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.

(Refer Slide Time: 16:45)

Formulation of Equations

$$\frac{dp_y}{dy} = \frac{p_y}{H-y} (1 + aK) + \gamma (1 - k_v - bk_h)$$

$$p_x = K \left[\frac{\gamma (1 - k_v - bk_h)}{2 + aK} \left\{ \frac{H^{(2+aK)}}{(H-y)^{(1+aK)}} - (H-y) \right\} \right]$$

$$a = \frac{(\tan \phi - \cot \theta)(1 + \tan \alpha \tan \delta)}{(\tan \alpha + \cot \theta)(1 + \tan \phi \cot \theta)} + \frac{(\tan \delta - \tan \alpha)}{(\tan \alpha + \cot \theta)}$$

$$b = \cot (\theta + \phi)$$

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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.

(Refer Slide Time: 17:08)

Seismic Passive Earth Pressure Coefficients

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


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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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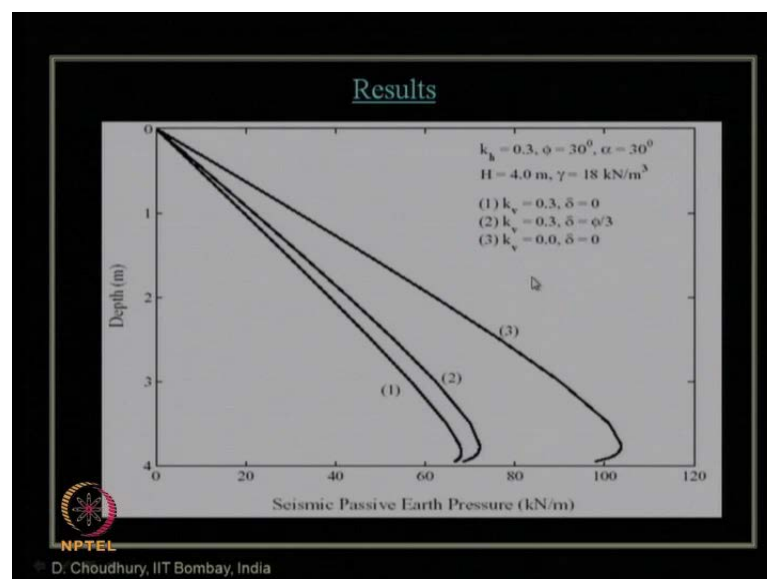
Point of Application
of Seismic Passive Earth Resistance

Case	ϕ (degree)	k_h, k_v		
		0.0, 0.0	0.3, 0.0	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


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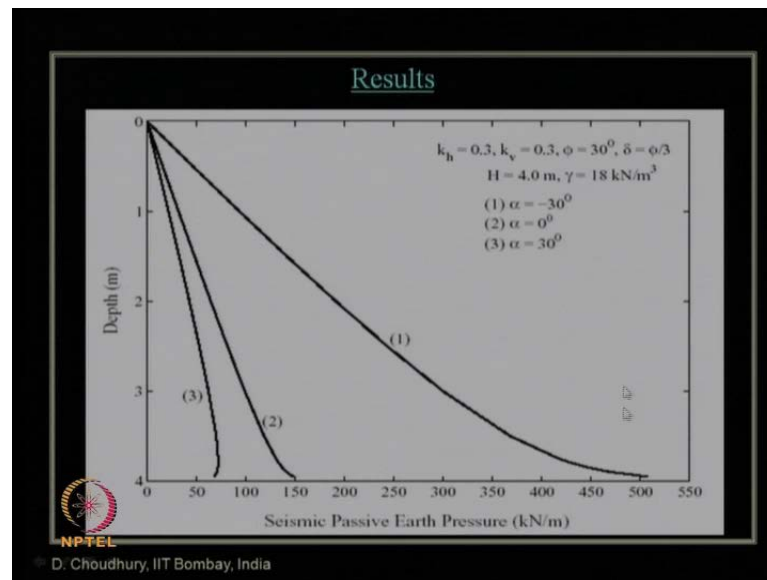
In this case, you can see the point of application for static case. It is always one-third from the height which is all known to us. But in seismic case it is not at one-third, but some other different height depending on the combinations of k_h, k_v .

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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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Pseudo-static Method

- **Richards and Elms (1979)** proposed a method for seismic design of **gravity retaining walls** which is based on **permanent** wall displacements. (**Displacement based approach**)

Seismic forces on gravity retaining wall
Richards and Elms (1979)

Displacement should be calculated by following formula, and should be checked against allowable displacement.

$$d_{pem} = 0.087 \frac{v_{max}^2 a_{max}^3}{a_y^4}$$

where, v_{max} is the peak ground velocity, a_{max} is the peak ground acceleration, a_y is the yield acceleration for the wall-backfill system.

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 Richards, R. Jr., and Elms, D. G. (1979). Seismic behavior of gravity retaining walls. *Journal of Geotechnical Engineering, ASCE, 105*: 4, 449-469. 37


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.

(Refer Slide Time: 18:17)

Pseudo-static Method

- ◎ Major limitations
 - > Representation of the **complex, transient, dynamic** effects of earthquake shaking by **single constant unidirectional pseudo-static** acceleration is **very crude**.
 - > Relation between K and the maximum ground acceleration is **not clear** i.e. 1.9 g acceleration does not mean $K = 1.9$
- ◎ Advantages
 - > **Simple** and **straight-forward**
 - > **No** advanced or complicated analysis is necessary.

It uses **limit state equilibrium** analysis which is routinely conducted by Geotechnical Engineers.

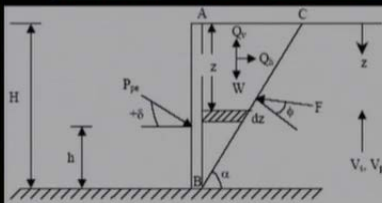


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We also discussed in our previous lecture, what are the major limitations about pseudo static approach like, it is very crude to approximate the actual complex nature of the dynamic problem and transient nature of the earthquake shaking into a single 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.

(Refer Slide Time: 18:49)

Development of Modern Pseudo-Dynamic Approach



Pseudo-Dynamic Approach by Steedman and Zeng (1991), Choudhury and Nimbalkar (2005)


$$a_h(z, t) = \{1 + (H - z) \cdot (I_s - 1) / H\} a_0 \sin [\omega \{t - (H - z) / V_s\}]$$

$$a_v(z, t) = \{1 + (H - z) \cdot (I_s - 1) / H\} a_0 \sin [\omega \{t - (H - z) / V_p\}]$$

Soil amplification is considered.
Frequency of earthquake excitation is considered.

Advantages

- Time duration of earthquake is considered.
- Phase differences between different waves can be considered.
- Amplitude of equivalent PGA can be considered.
- Considers seismic body wave velocities traveling during earthquake.



Choudhury and Nimbalkar, (2005), in *Geotechnique*, ICE Vol. 55(10), pp. 949-953.

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.

(Refer Slide Time: 19:52)

The coefficient of seismic passive resistance (K_{pe}) is given by,

$$K_{pe} = \frac{1}{\tan \alpha \cos(\delta + \phi + \alpha)} - \frac{k_h}{2\pi^2 \tan \alpha} \left(\frac{TV_s}{H} \right) \\ \times \frac{\cos(\alpha + \phi)}{\cos(\delta + \phi + \alpha)} \times m_1 - \frac{k_v}{2\pi^2 \tan \alpha} \left(\frac{TV_p}{H} \right) \\ \times \frac{\sin(\alpha + \phi)}{\cos(\delta + \phi + \alpha)} \times m_2$$

where

$$m_1 = 2\pi \cos 2\pi \left(\frac{t}{T} - \frac{H}{TV_s} \right) + \left(\frac{TV_s}{H} \right) \\ \times \left[\sin 2\pi \left(\frac{t}{T} - \frac{H}{TV_s} \right) - \sin 2\pi \left(\frac{t}{T} \right) \right] \quad \text{and} \quad m_2 = 2\pi \cos 2\pi \left(\frac{t}{T} - \frac{H}{TV_p} \right) + \left(\frac{TV_p}{H} \right) \\ \times \left[\sin 2\pi \left(\frac{t}{T} - \frac{H}{TV_p} \right) - \sin 2\pi \left(\frac{t}{T} \right) \right]$$

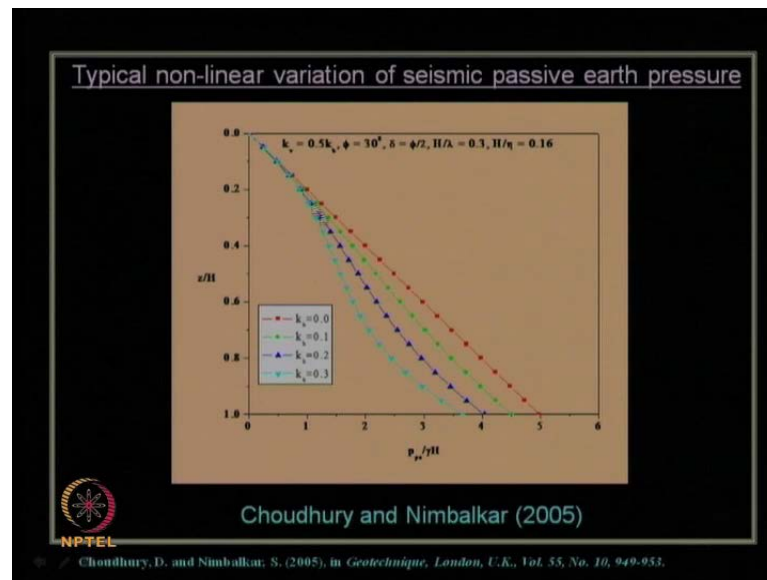
The seismic passive earth pressure distribution is given by,

$$p_{pe}(t) = \frac{dp_{pe}(t)}{dz} = \frac{\gamma z}{\tan \alpha \cos(\alpha + \delta + \phi)} \frac{\sin(\alpha + \phi)}{\cos(\alpha + \delta + \phi)} \\ - \frac{k_h \gamma z}{\tan \alpha \cos(\alpha + \delta + \phi)} \frac{\cos(\alpha + \phi)}{\cos(\alpha + \delta + \phi)} \sin \omega \left(t - \frac{z}{V_s} \right) \\ - \frac{k_v \gamma z}{\tan \alpha \cos(\alpha + \delta + \phi)} \frac{\sin(\alpha + \phi)}{\cos(\alpha + \delta + \phi)} \sin \omega \left(t - \frac{z}{V_p} \right)$$

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Choudhury, D. and Nimbalkar, S. (2005), in *Geotechnique*, London, U.K., Vol. 55, No. 10, 949-953.

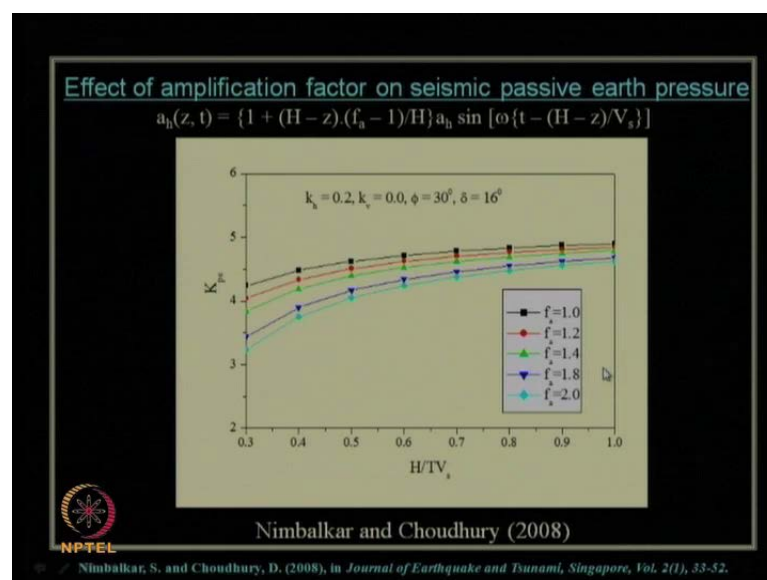
And, we had proposed this closed form design solution or the designer's equation, which is necessary to design an seismic passive earth pressure. In case of retaining wall, the seismic passive earth pressure coefficient expression K_{pe} which needs to be optimized with respect to this alpha failure 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_{pe} .

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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.

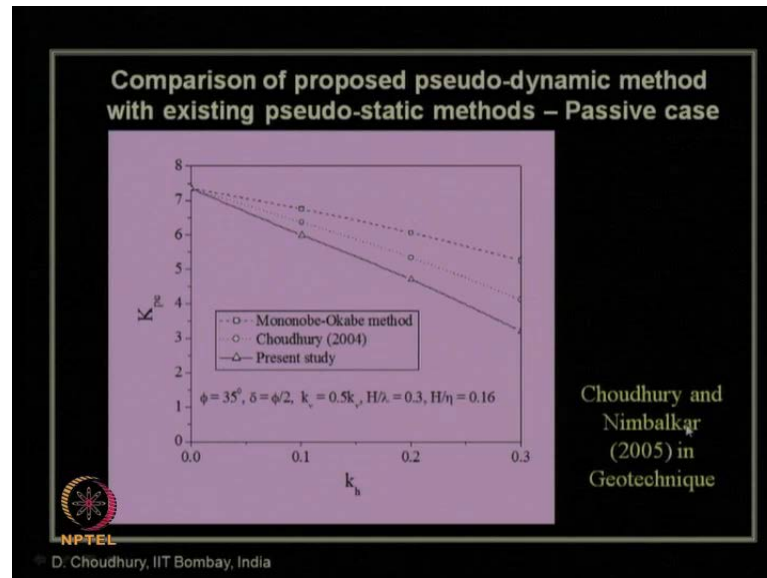
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And, the effect of this soil amplification factor can be seen on the seismic passive earth pressure coefficient. As the soil amplification factor increases, there is a decrease,

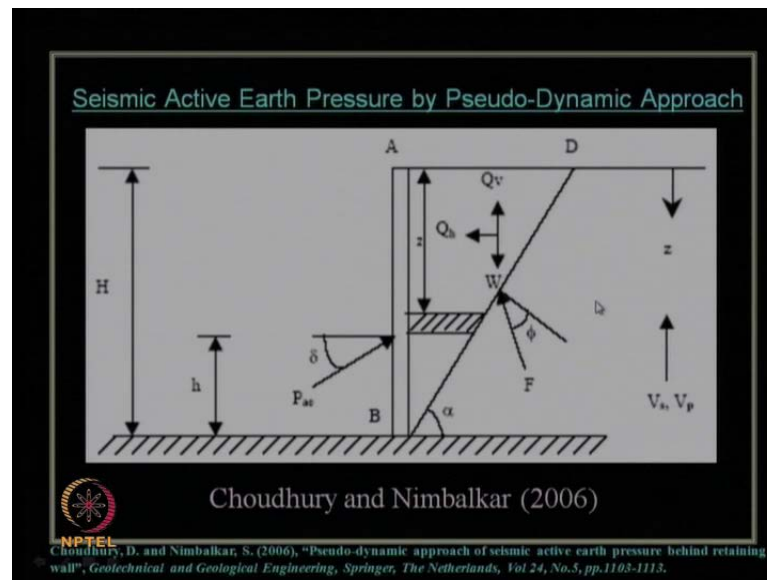
significant decrease in this seismic passive earth pressure coefficient value which needs to be considered in the design; which is available in journal paper “Nimbalkar and Choudhury 2008” in the journal of “Earthquake and Tsunami”.

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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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$a_h(z, t) = a_h \sin [\omega \{t - (H - z)/V_s\}]$ and $a_v(z, t) = a_v \sin [\omega \{t - (H - z)/V_p\}]$
 where ω = angular frequency; t = time elapsed; V_s = shear wave velocity;
 V_p = primary wave velocity.

$$Q_h(t) = \int_0^H m(z) a_h(z, t) dz = \frac{\lambda \gamma a_h}{4 \pi^2 g \tan \alpha} [2 \pi H \cos \omega \zeta + \lambda (\sin \omega \zeta - \sin \omega t)]$$

where, $\lambda = TV_s$ is the wavelength of the vertically propagating shear wave and $\zeta = t - H/V_s$.

$$Q_v(t) = \int_0^H m(z) a_v(z, t) dz = \frac{\eta \gamma a_v}{4 \pi^2 g \tan \alpha} [2 \pi H \cos \omega \psi + \lambda (\sin \omega \psi - \sin \omega t)]$$

where, $\eta = TV_p$ is the wavelength of the vertically propagating primary wave and $\psi = t - H/V_p$.

The total (static plus dynamic) active thrust is given by,

$$P_{ae}(t) = \frac{W \sin(\alpha - \phi) + Q_h(t) \cos(\alpha - \phi) - Q_v(t) \sin(\alpha - \phi)}{\cos(\delta + \phi - \alpha)}$$

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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 $P_a e$ can be obtained like this.

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The seismic active earth pressure coefficient, K_{ae} is defined as

$$K_{ae} = \frac{1}{\tan \alpha} \frac{\sin(\alpha - \phi)}{\cos(\delta + \phi - \alpha)} + \frac{k_x}{2\pi^2 \tan \alpha} \left(\frac{TV_v}{H} \right) \times \frac{\cos(\alpha - \phi)}{\cos(\delta + \phi - \alpha)} \times m_1 + \frac{k_z}{2\pi^2 \tan \alpha} \left(\frac{TV_v}{H} \right) \times \frac{\sin(\alpha - \phi)}{\cos(\delta + \phi - \alpha)} \times m_2$$

where,

$$m_1 = \left[2\pi \cos 2\pi \left(\frac{t}{T} - \frac{H}{TV_v} \right) + \left(\frac{TV_v}{H} \right) \left(\sin 2\pi \left(\frac{t}{T} - \frac{H}{TV_v} \right) - \sin 2\pi \left(\frac{t}{T} \right) \right) \right]$$

$$m_2 = \left[2\pi \cos 2\pi \left(\frac{t}{T} - \frac{H}{TV_v} \right) + \left(\frac{TV_v}{H} \right) \left(\sin 2\pi \left(\frac{t}{T} - \frac{H}{TV_v} \right) - \sin 2\pi \left(\frac{t}{T} \right) \right) \right]$$

The seismic active earth pressure distribution is given by,

$$p_{ae}(z) = \frac{\delta P_{ae}(z)}{\delta z} = \frac{\gamma z}{\tan \alpha} \frac{\sin(\alpha - \phi)}{\cos(\delta + \phi - \alpha)} + \frac{k_x \gamma z}{\tan \alpha} \frac{\cos(\alpha - \phi)}{\cos(\delta + \phi - \alpha)} \sin \left[w \left(t - \frac{z}{V_v} \right) \right] + \frac{k_z \gamma z}{\tan \alpha} \frac{\sin(\alpha - \phi)}{\cos(\delta + \phi - \alpha)} \sin \left[w \left(t - \frac{z}{V_v} \right) \right]$$

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Choudhury and Nimbalkar (2006)

And, seismic active earth pressure coefficient K_{ae} 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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Typical non-linear variation of seismic active earth pressure

Choudhury and Nimbalkar (2006), in *Geotechnical and Geological Engineering*, 24(5), 1103-1113.

$$a_{01}(z, t) = \{1 + (H - z) \cdot (f_1 - 1)\} H_1 a_0 \sin [\omega \{t - (H - z) / V_v\}]$$

Effect of soil amplification on seismic active earth pressure

Nimbalkar and Choudhury (2008), in *Journal of Earthquake and Tsunami*, 2(1), 33-52.

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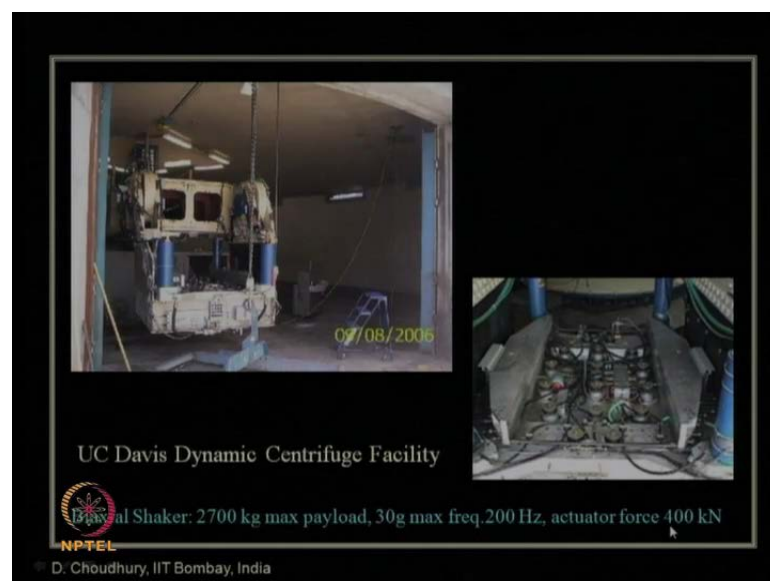
The results obtained by research, other researchers were used. They did an analysis for this BART. BART is nothing but bay area rapid transit or a metro railway system. For underground tunnel, they need to redesign it under different earthquake condition; latest earthquake condition.

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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 capacity 240 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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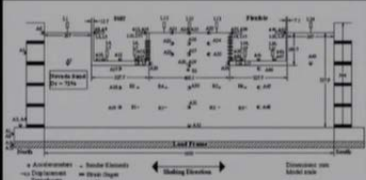
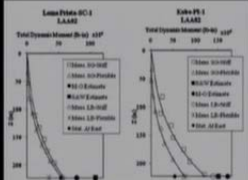



You can see, it can go up to 30 g maximum frequency of 200 hertz with actuator force of 400 Newton. This is a biaxial shaker which is implanted on this box of this centrifuge container position, where it can be kept.

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Dynamic Centrifuge Tests

- **Atik and Sitar (2010)** have conducted an **experimental and analytical program** was designed and conducted to evaluate the magnitude and distribution of seismically induced lateral earth pressures on **cantilever retaining structures with dry medium dense sand backfill**.

 Model used for centrifuge test by Atik and Sitar (2010) Typical results as given by Atik and Sitar (2010)

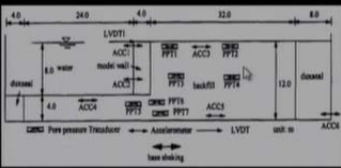
NPTEL, L. and N. Sitar (2010). Seismic Earth Pressures on Cantilever Retaining Structures. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 136-10, 1324-1333.* 52

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.

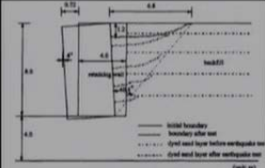
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Dynamic Centrifuge Test Results

- Zeng (1998) described the behaviour of gravity quay walls under earthquake loading using data from three centrifuge tests.
- Using a modified pseudo-static approach, ground settlement in the backfill, influence of pore pressure on the wedge angle has also been studied.



Model prepared for centrifuge test by Zeng (1998)



Typical results obtained by Zeng (1998)

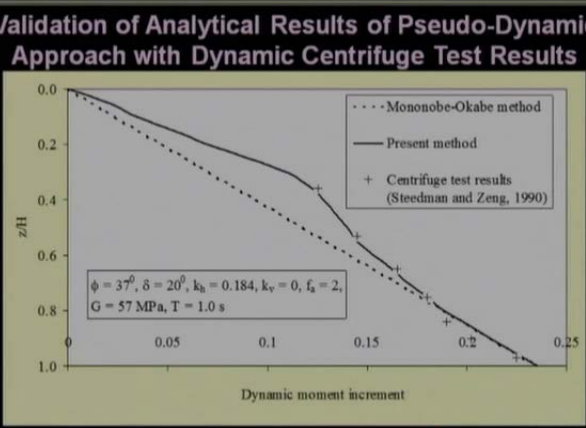
NPTEL © 2012. X. (1998). Seismic response of gravity quay walls I: Centrifuge modelling. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 124:5, 406-417.* 53

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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Validation of Analytical Results of Pseudo-Dynamic Approach with Dynamic Centrifuge Test Results



$\phi = 37^\circ, \delta = 20^\circ, k_h = 0.184, k_v = 0, f_d = 2,$
 $G = 57 \text{ MPa}, T = 1.0 \text{ s}$

Dynamic moment increment, $\sqrt[3]{\frac{M}{\gamma H^3}}$, where $M(Z, t) = \int_0^Z p_{ac}(z, t) \cos \delta(Z-z) dz$

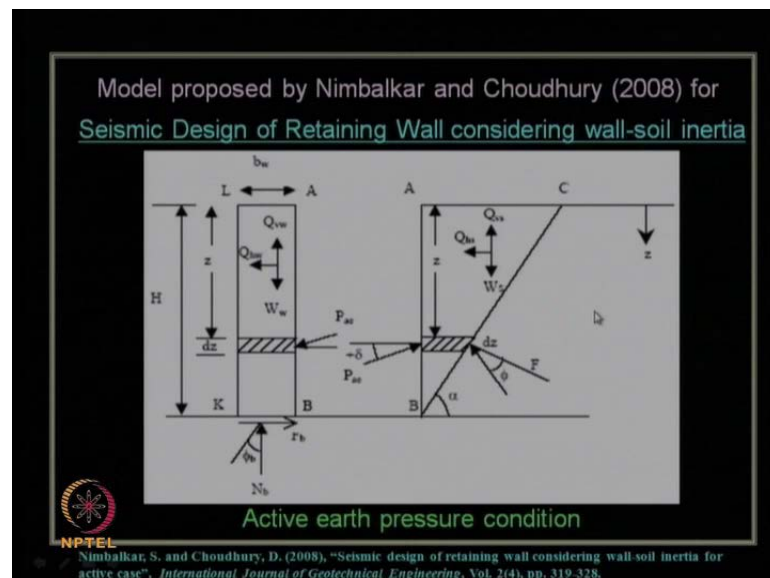
NPTEL © 2012. D. Choudhury, IIT Bombay

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.

(Refer Slide Time: 27:34)



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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Proposed Design Factors for Retaining Wall
by Nimbalkar and Choudhury (2008)

Soil thrust factor, $F_T = \frac{K_{ae}}{K_a}$

Wall inertia factor, $F_I = \frac{C_{IE}(t)}{C_{Ia}}$

where, $C_{IE}(t) = \frac{\cos \delta - \sin \delta \tan \phi_b}{\tan \phi_b} + \frac{Q_{Ie}(t) + Q_{Ie}(t) \tan \phi_b}{P_{Ie}(t) \tan \phi_b}$

$C_{Ia} = \frac{\cos \delta - \sin \delta \tan \phi_b}{\tan \phi_b}$

Combined dynamic factor, $F_w = F_T F_I = \frac{W_w(t)}{W_w}$

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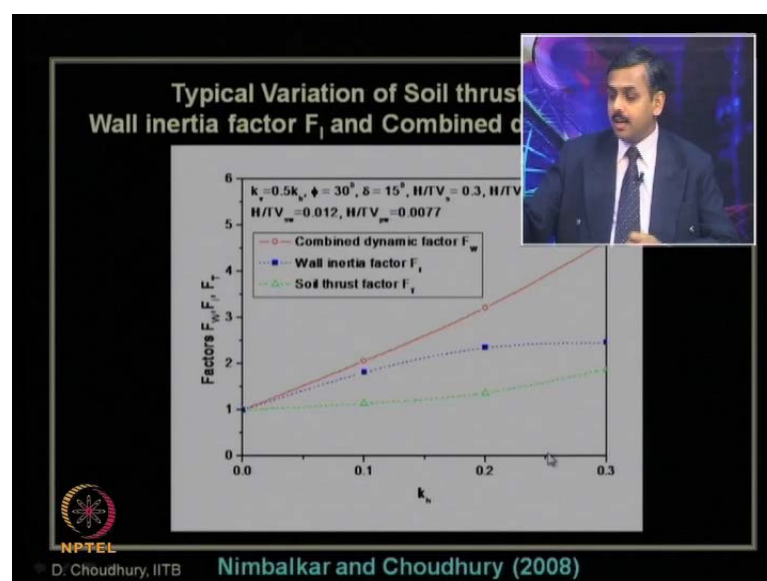
From the analysis, it has been proposed one three different factors were proposed. One is soil thrust factor, which is nothing but ratio of that seismic active earth pressure to the static active earth pressure; another is wall inertia factor, which is given by this C I e which is the function of time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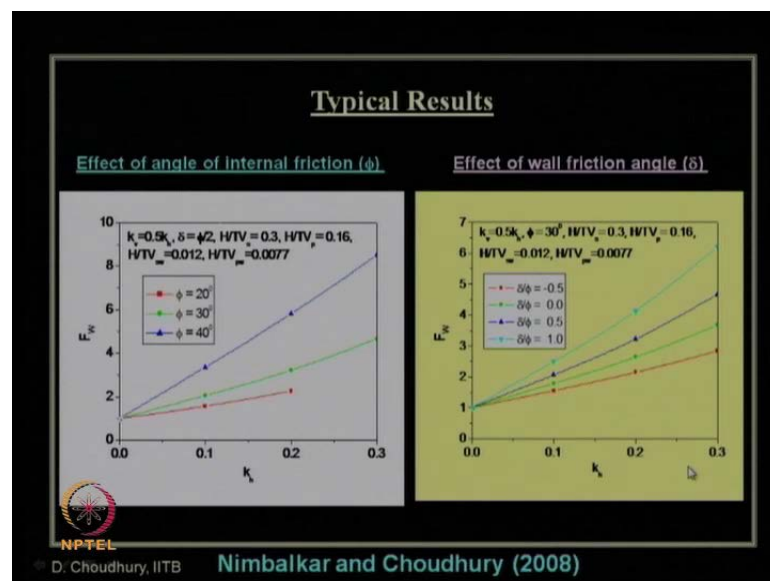


Let me clarify it once again. If we look at this variation of this proposed design factor what we have proposed here in this paper, this red color line gives us the combined dynamic factor F_w .

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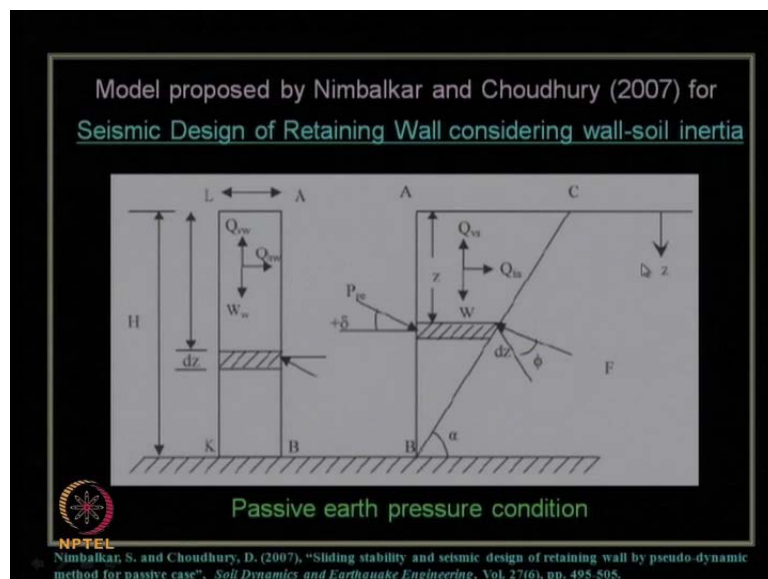
Comparison of Soil thrust factor F_T , Wall inertia factor F_I and Combined Dynamic Factor F_w

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	
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
	0.00	2.493	2.021	5.039	2.382	3.255	7.753
0.4	0.00	3.500	1.909	6.683	3.223	7.464	24.059

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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”. This F_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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Proposed Design Factors for Retaining Wall
by Nimbalkar and Choudhury (2007)

$$F_T = \frac{K_{ae,pe}}{K_{a,p}}$$

$$F_I = \frac{C_{IE}(t)}{C_{ia,lp}}$$

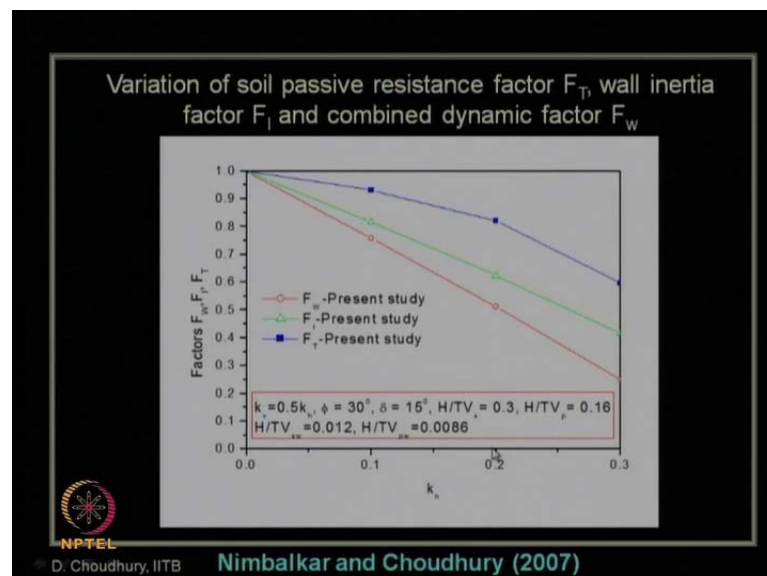
$$C_{ia,lp} = \pm \frac{\cos \delta - \sin \delta \tan \phi_b}{\tan \phi_b}$$

$$F_w = F_T F_I = \frac{W_w(t)}{W_w}$$


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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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Pseudo-dynamic Method in Displacement –based analysis

- Choudhury and Nimbalkar (2007) proposed pseudo-dynamic method to compute the seismic rotational displacements of retaining wall for passive earth pressure condition. (*Soil Dynamics and Earthquake Engg., 2007*)

(a) Pseudo-dynamic forces acting on retaining wall system for rotational stability

Variation of rotational displacement (θ) with k_h

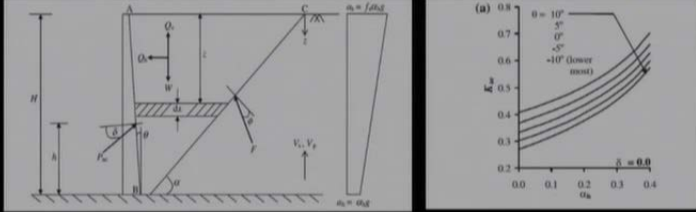
Choudhury, D. and Nimbalkar, S. (2007). Seismic rotational displacement of gravity walls by pseudo-dynamic method: Passive case. *Soil Dynamics and Earthquake Engineering*, 27, 242–249.

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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Pseudo-dynamic Method

- Ghosh (2008) presented study on seismic active earth pressure behind a non-vertical cantilever retaining wall using pseudo-dynamic analysis.



The diagram on the left shows a cross-section of a non-vertical cantilever retaining wall. The wall has a total height H and a base width b . The wall is inclined at an angle θ to the vertical. The soil behind the wall is represented by a trapezoidal shape with a top width w and a bottom width d . The soil surface is at a height z from the base. The wall is subjected to a horizontal seismic force Q_h and a vertical seismic force Q_v . The active earth pressure is shown as a triangular distribution with a resultant force P_a acting at a distance z from the base. The wall is supported by a base with a reaction force V_r and a moment M_r . The soil surface is at a height z from the base. The wall is subjected to a horizontal seismic force Q_h and a vertical seismic force Q_v . The active earth pressure is shown as a triangular distribution with a resultant force P_a acting at a distance z from the base. The wall is supported by a base with a reaction force V_r and a moment M_r .

Model considered by Ghosh (2008) by pseudo-dynamic method for active case

Variation of active pressure coefficient K_{ae} with α_h for $f = 30^\circ$, $\Psi\lambda = 0.3$, $H/\eta = 0.16$, $f_a = 1$ by Ghosh (2008)

Ghosh P., (2008), "Seismic active earth pressure behind a non-vertical retaining wall using pseudo-dynamic analysis", *Canadian Geotechnical Journal*, 45, 17-123.

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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 K_a e design charts based on different wall inclination. You can see, we had proposed for wall inclination of 0. This θ , it is with respect to vertical as you can see from this picture. So, as wall inclination changes the value of K_a e also changes as proposed by Ghosh in 2008.

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Pseudo-dynamic Method

- **Basha and Babu (2010)** have presented use of **pseudo-dynamic** method to compute the **rotational** displacements of **gravity** retaining walls under **passive condition** when subjected to seismic loads.
- Authors have combined the concept of **Newmark's** sliding block method (1965) for computing the **rotational** displacements under **seismic** condition by using the **limit equilibrium** analysis under seismic conditions.
- Major conclusion was that **major** factor which controls the **amount of rotation** of wall during an earthquake is **the soil friction angle**.

Basha, B. and Babu, G. (2010). Seismic rotational displacements of gravity walls by pseudodynamic method with curved rupture surface. *Int. J. Geomech., ASCE, 10:3, 93-105.*



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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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Pseudo-dynamic Method

- Ghosh and Sharma (2010) presented the **pseudo-dynamic** analysis for calculating **seismic active** earth pressure for non-vertical retaining wall supporting **c- ϕ** backfill.
- Bellezza et al. (2012) claim that a **more** rational pseudo-dynamic approach has been developed for **fully submerged** soil under the assumption that a restrained or free water condition exists within the backfill.
- Bellezza et al. (2012) have also extended their study to study the effect of **amplification** phenomena.

Ghosh, S. and Sharma, R. P. (2010). "Pseudo-dynamic active response of non-vertical retaining wall supporting c- ϕ backfill." *Geotechnical and Geological Engineering, 28:5, 633-641.*

Bellezza, I, D'Alberto, D, Fentini, R (2012). Pseudo-dynamic approach for active thrust of submerged soils. *Proceedings of the ICE - Geotechnical Engineering, 165:5, 321-333.*



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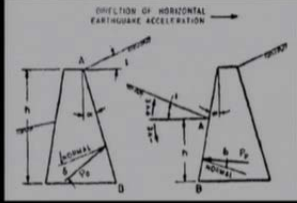
Other recent researcher like Ghosh and Sharma in 2010, they also use the pseudo dynamic model for seismic active earth pressure. For a non-vertical retaining wall, what was the addition they made? They used the C phi backfill. That is, first initially we had proposed it for only cohesion less backfill. 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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Provisions in Design Codes

- **Indian Design Code**
 - **IS 1893 - Part 5 (1984)**, provides information regarding **earthquake resistant** design for retaining wall for **active and passive** case. Use of M-O method.
 - Point of application at mid-height for dynamic component.
 - Pseudo-static is used, which **excludes** the **deformation** criteria.



IS 1893, Indian Standard Criteria for Earthquake Resistant Design of Structures. Part 5 (fourth revision), 1984.

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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 latest 1984 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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Codal Provisions

- Indian Design Code
- As per IS 1893 - Part 5 (1984), active earth pressure exerted against wall can be,

$$P_a = (1/2) W. H^2 C_a$$
- where C_a is given by,

$$C_a = \frac{(1 \pm \alpha_v) \cos^2(\phi - \lambda - \alpha)}{\cos \lambda \cos^2 \alpha \cos(\delta + \lambda + \alpha)} \times \left[\frac{1}{1 + \left[\frac{\sin(\phi + \delta) \sin(\phi - \lambda - \delta)}{\cos(\alpha - \lambda) \cos(\delta + \alpha + \lambda)} \right]^{1/2}} \right]^2$$

where, α_v vertical seismic coefficient - its direction being taken consistently throughout the stability analysis of wall and equal to $(1/2) \alpha_h$ - where α_h horizontal seismic coefficient.

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And, what are the steps? This is the formula needs to be used P_a 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 α_v is the vertical acceleration coefficient, which should be taken as half of that α_h ; that is, horizontal seismic acceleration coefficient. So, this is with respect to the passive case.


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Codal Provisions

- Indian Design Code
- IS 1893 - Part 5 (1984), passive earth pressure exerted against wall can be,
 $P_p = (1/2) W. H^2 C_p$
- where C_p is given by,

$$C_p = \frac{(1 + \alpha_v) \cos^2(\delta + \alpha - \lambda)}{\cos \lambda \cos^2 \alpha \cos(\delta - \alpha + \lambda)} \times \left[\frac{1}{1 + \left\{ \frac{\sin(\phi + \delta) \sin(\phi + i - \delta)}{\cos(\alpha - i) \cos(\delta - \alpha + \lambda)} \right\}^{1/2}} \right]^2$$

where f is soil friction angle, δ friction angle for soil and wall


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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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Provisions in Design Codes

- **European Design Code**
 - Eurocode 8 (2003) explains the design of structures for earthquake resistance, wherein part 5 explains the procedure for foundations, retaining structures and geotechnical aspects.
 - It is based on pseudo-static method and follows displacement (for translation and rocking mode) based approach given by Richards and Elms (1979).
 - Eurocode 8 (2003) highlights guidelines to take into account values of k_h and k_v in absence of any study.

 NPTEL Eurocode 8, EN 1998, Design provisions for earthquake resistance of structures. 2003. D. Choudhury, IIT Bombay, India 70

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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Codal Provisions

- Eurocode 8 (2003) mentions -
 1. For the purpose of the **pseudo-static** analysis, the seismic action shall be **represented** by a **set of** horizontal and vertical static **forces equal to the product** of the gravity forces and a seismic coefficient.
 2. The **vertical** seismic action shall be considered as acting upward or downward so as to produce the most **unfavourable** effect.
 3. The **intensity** of such equivalent seismic forces depends, for a **given seismic zone**, on the **amount of permanent displacement** which is both acceptable and **actually permitted** by the adopted structural solution.

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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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Codal Provisions

- Eurocode 8 (2003)
 - It mentions that, in the **absence** of specific studies, the horizontal (k_h) and vertical (k_v) seismic coefficients affecting all the masses shall be taken as-

$$k_h = \alpha \frac{S}{r}$$
$$k_v = \pm 0.5k_h, \text{ if } a_{vg}/a_g \text{ is larger than } 0.6$$
$$k_v = \pm 0.33k_h, \text{ otherwise}$$

where, k_h and k_v are seismic horizontal and vertical coefficients, α ratio of the design ground acceleration on type A ground, a_g , to the acceleration of gravity g , a_{vg} is design ground acceleration in the vertical direction, a_g is design ground acceleration on type A ground.

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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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Provisions in Design Codes

- **International Building Code**
 - **IBC (2006)** categorizes sites into categories namely **A, B, C, D, E, F** based on soil profile, **shear wave velocity, SPT values and undrained shear strength values.**
 - Based on that, the design **seismic category** should be selected.
 - It mentions that retaining walls shall be designed to ensure stability against **overturning, sliding, excessive foundation pressure and water uplift.**

International Building Code (2006),
INTERNATIONAL CODE COUNCIL, INC.

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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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Requalification of Geotechnical Earth Retaining Structures

- Eurocode 8 (2003) mentions –
 1. Earth retaining structures shall be designed to **fulfil their function during and after an earthquake, without** suffering significant structural damage
 2. **Permanent displacements**, in the form of combined sliding and tilting, the latter due to irreversible deformations of the foundation soil, may be acceptable if it is shown that they are **compatible** with functional and/or aesthetic requirements.

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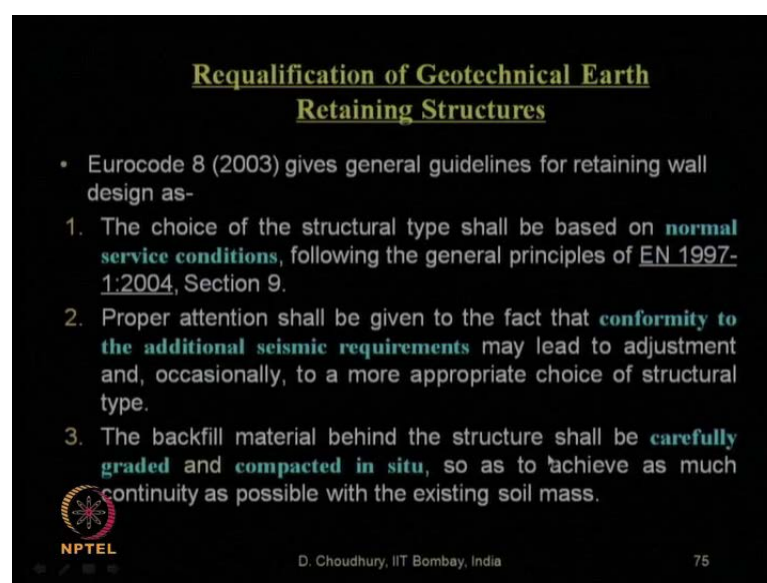
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And for that, what is proposed? The Eurocode mentions, so if we do the requalification of geotechnical earth retaining structures under this seismic condition like Eurocode suggests that, retaining factors should be designed to fulfill their function 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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Requalification of Geotechnical Earth Retaining Structures

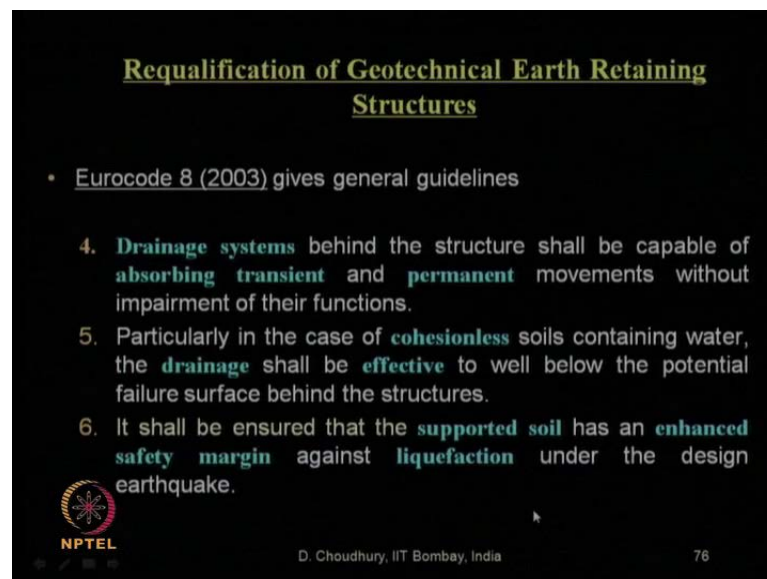
- Eurocode 8 (2003) gives general guidelines for retaining wall design as-
 1. The choice of the structural type shall be based on **normal service conditions**, following the general principles of EN 1997-1:2004, Section 9.
 2. Proper attention shall be given to the fact that **conformity to the additional seismic requirements** may lead to adjustment and, occasionally, to a more appropriate choice of structural type.
 3. The backfill material behind the structure shall be **carefully graded** and **compacted in situ**, so as to achieve as much continuity as possible with the existing soil mass.

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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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Requalification of Geotechnical Earth Retaining Structures

- Eurocode 8 (2003) gives general guidelines
- 4. **Drainage systems** behind the structure shall be capable of **absorbing transient** and **permanent** movements without impairment of their functions.
- 5. Particularly in the case of **cohesionless** soils containing water, the **drainage** shall be **effective** to well below the potential failure surface behind the structures.
- 6. It shall be ensured that the **supported soil** has an **enhanced safety margin** against **liquefaction** under the design earthquake.

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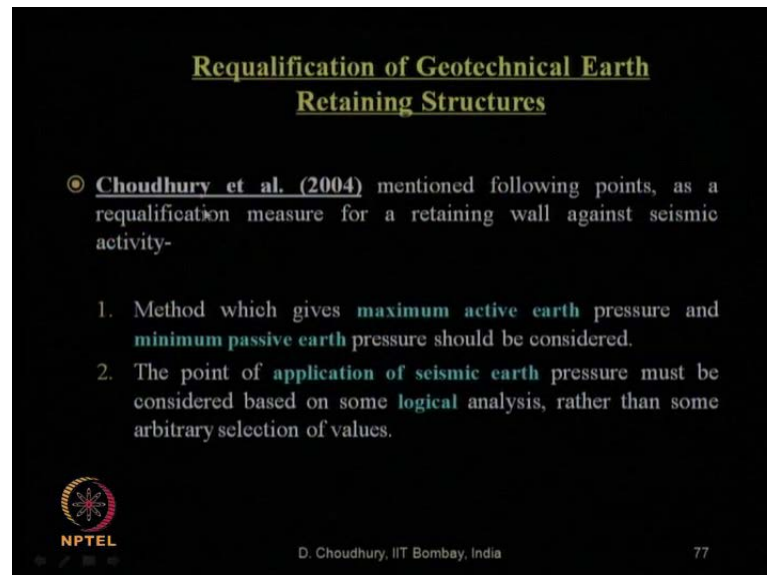
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Like drainage system behind the structure, should always be present. Because if you provide the drainage there will be no extra water pressure coming on the wall. As we all know in the static case of wall design, the same thing also acts here for the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

- **Choudhury et al. (2004)** mentioned following points, as a requalification measure for a retaining wall against seismic activity-

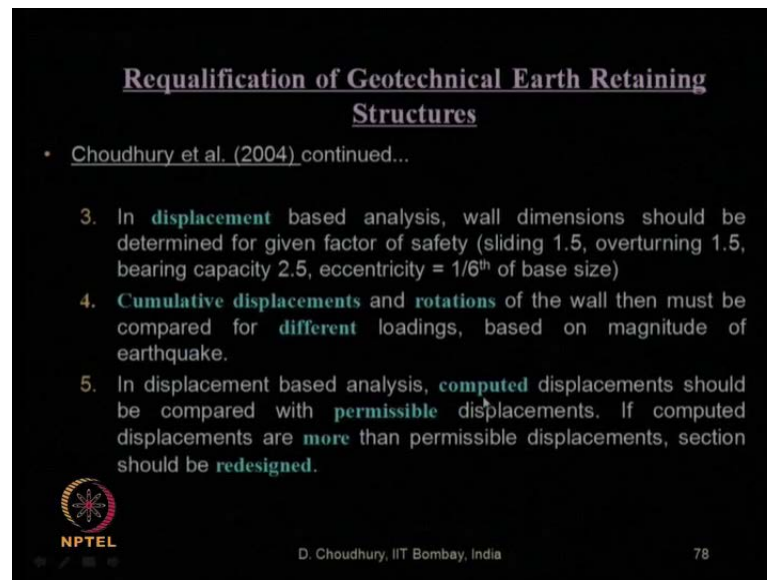
1. Method which gives **maximum active earth** pressure and **minimum passive earth** pressure should be considered.
2. The point of **application of seismic earth** pressure must be considered based on some **logical analysis**, rather than some arbitrary selection of values.

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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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Requalification of Geotechnical Earth Retaining Structures

- Choudhury et al. (2004) continued...
- 3. In **displacement** based analysis, wall dimensions should be determined for given factor of safety (sliding 1.5, overturning 1.5, bearing capacity 2.5, eccentricity = 1/6th of base size)
- 4. **Cumulative displacements** and **rotations** of the wall then must be compared for **different** loadings, based on magnitude of earthquake.
- 5. In displacement based analysis, **computed** displacements should be compared with **permissible** displacements. If computed displacements are **more** than permissible displacements, section should be **redesigned**.

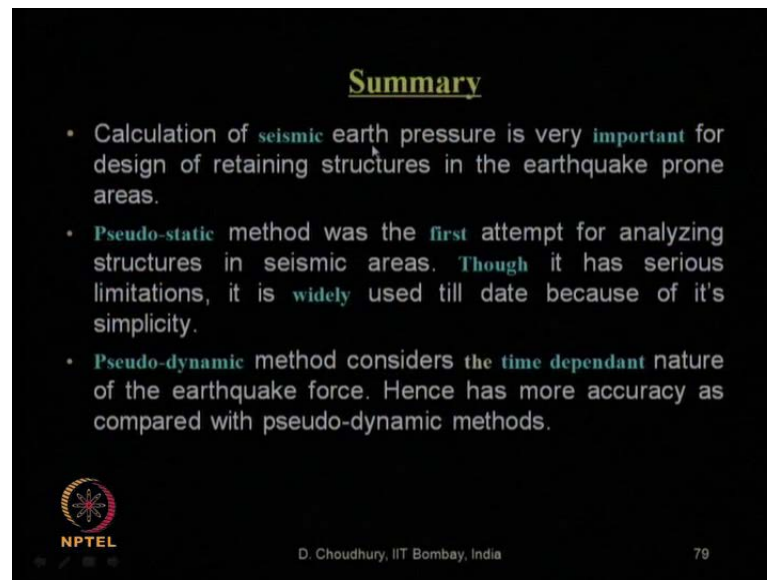
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Also the displacement based analysis mentions that, given factor of safety in static case has to be maintained in dynamic case as well. Like for sliding 1.5, overturning 1.5, bearing capacity 2.5 and eccentricity should be one-sixth of the base size. 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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Summary

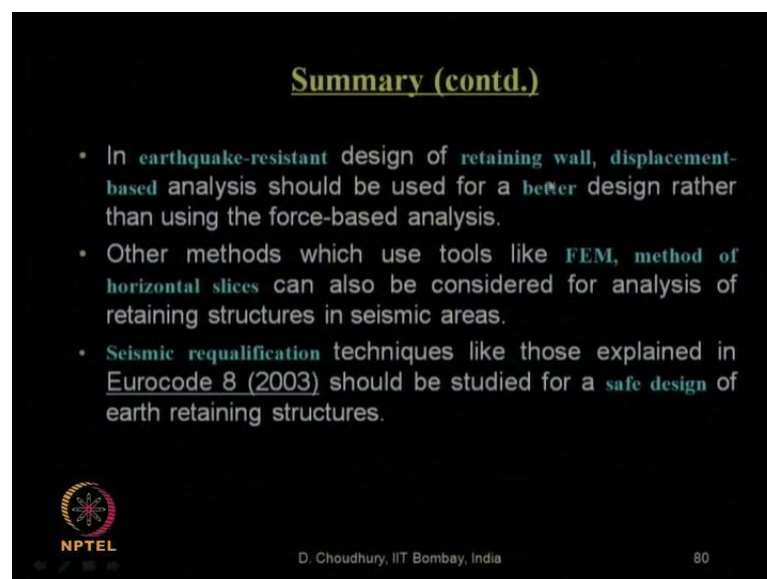
- Calculation of seismic earth pressure is very important for design of retaining structures in the earthquake prone areas.
- Pseudo-static method was the first attempt for analyzing structures in seismic areas. Though it has serious limitations, it is widely used till date because of its simplicity.
- Pseudo-dynamic method considers the time dependant nature of the earthquake force. Hence has more accuracy as compared with pseudo-dynamic methods.

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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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Summary (contd.)

- In earthquake-resistant design of retaining wall, displacement-based analysis should be used for a better design rather than using the force-based analysis.
- Other methods which use tools like FEM, method of horizontal slices can also be considered for analysis of retaining structures in seismic areas.
- Seismic requalification techniques like those explained in Eurocode 8 (2003) should be studied for a safe design of earth retaining structures.

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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.