Geotechnical Earthquake Engineering
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Module - 9
Lecture - 37
Seismic Analysis and Design of Various Geotechnical Structures (Contd…)

Let us start our today's lecture for 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, let me have a quick recap what we have learnt in our previous lecture, which was on the seismic design of retaining wall.

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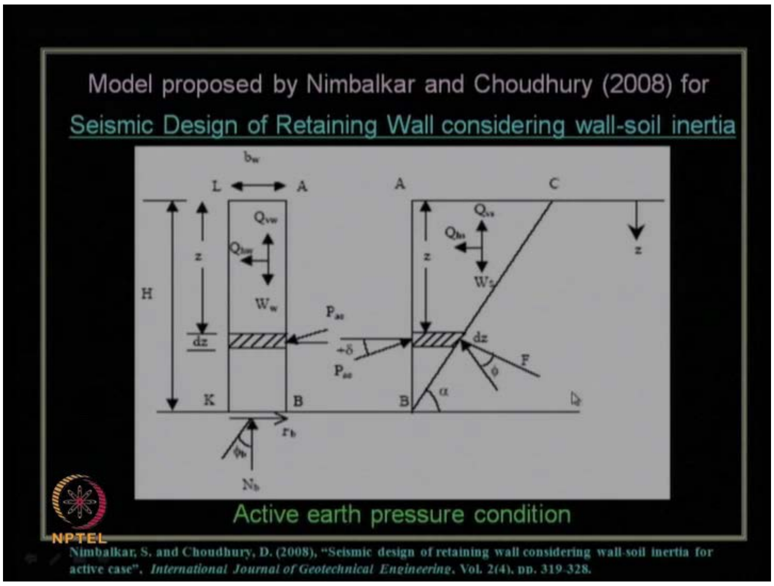

To recap quickly, in the previous lecture, we discussed about the displacement based design approach for the seismic design of retaining wall both under active state as well as passive state considering sliding as well as rotational stability is concerned. So, this paper, where we considered both the inertia forces acting on the wall as well as on the failure mass of the soil under active condition, that details we have given in this paper.

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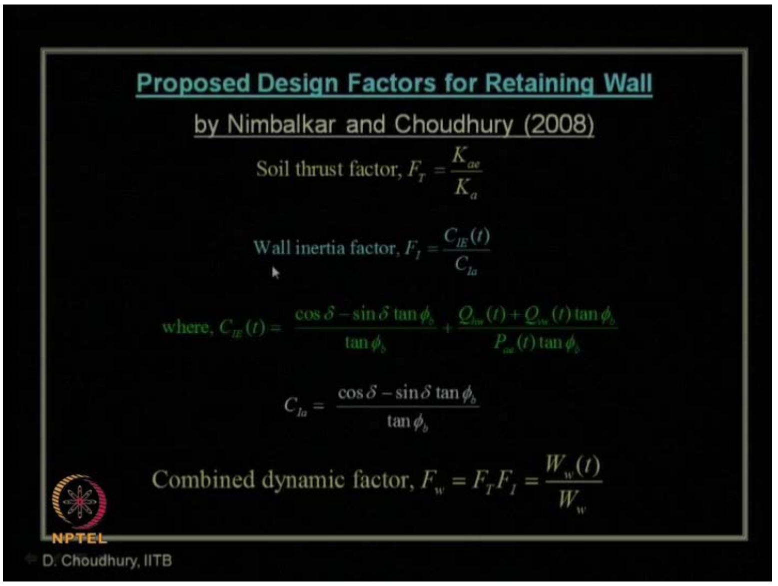

So, how to find out the weight of the wall required under earthquake condition to be designed; that has been proposed through this combined dynamic factor; which is nothing but ratio of the weight of the wall required under dynamic case by weight of the wall required for static case with respect to sliding stability. So, in static case, we know how to design it with factor of safety of 1.5.

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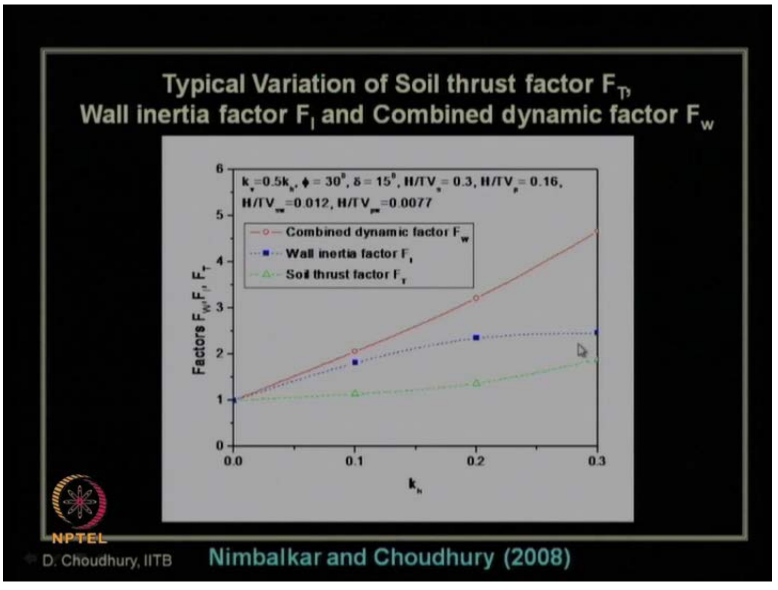

So, the same approach we will use and then the proposed design chart of this combined dynamic factor F W to be used for a seismic zone, where the k h value or the input seismic acceleration for the design will be known to us. Correspondingly, you will get the value of F W; once you get the F W, automatically, what you can do? You are getting this W W value, which further will help you to get the section of the wall, which will be stable with respect to that particular magnitude of earthquake and the design will be complete under the active state.

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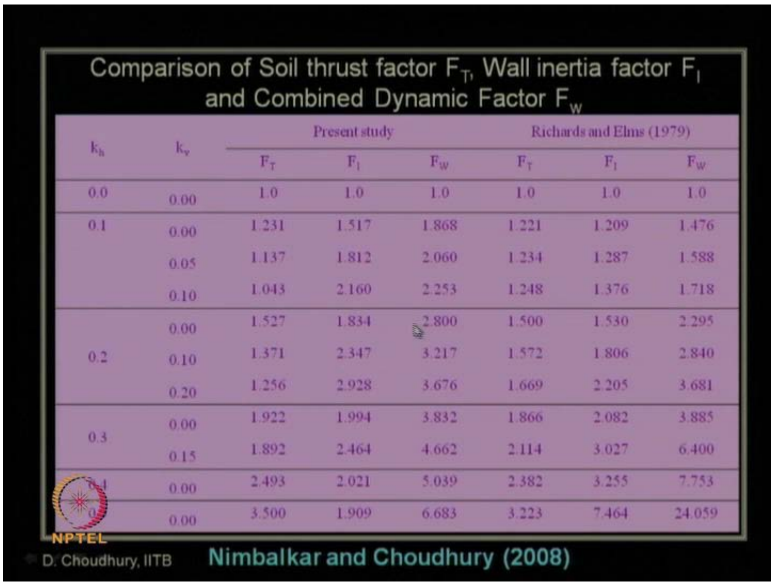

Comparison of Soil thrust factor F_T, Wall inertia factor F_I and Combined Dynamic Factor F_W

k_h	k_v	Present study F_T	Present study F_I	Present study F_W	Richards and Elms (1979) F_T	Richards and Elms (1979) F_I	Richards and Elms (1979) 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

And, we have compared our proposed value of pseudo-dynamic approach of this combined dynamic factor with the available results using pseudo-static approach as given by Richard and Elms. As I said that, this method is commonly used in euro code Richard and Elms displacement-based criteria. So, you can see the comparison given over here.

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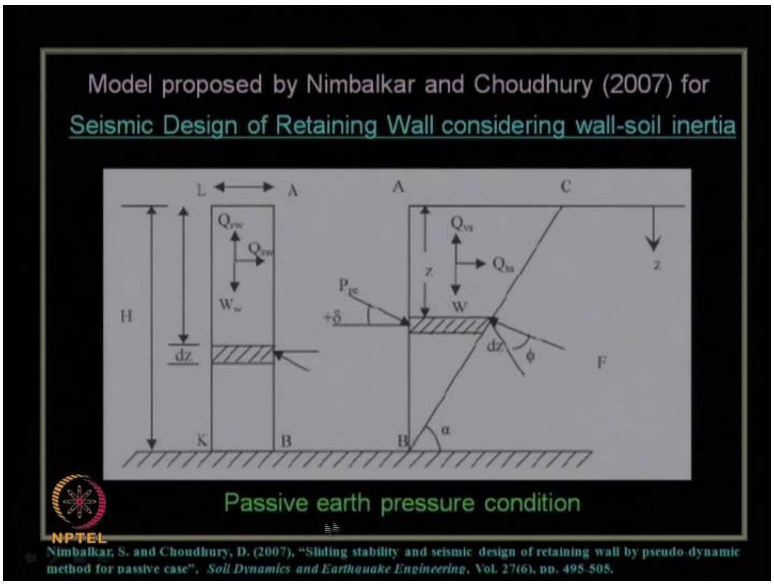

The similar approach has been extended for the passive state of earth pressure also for design of the retaining wall using pseudo-dynamic approach as can be obtained in this journal paper of Nimbalkar and Choudhury – Soil Dynamics and Earthquake Engineering journal.

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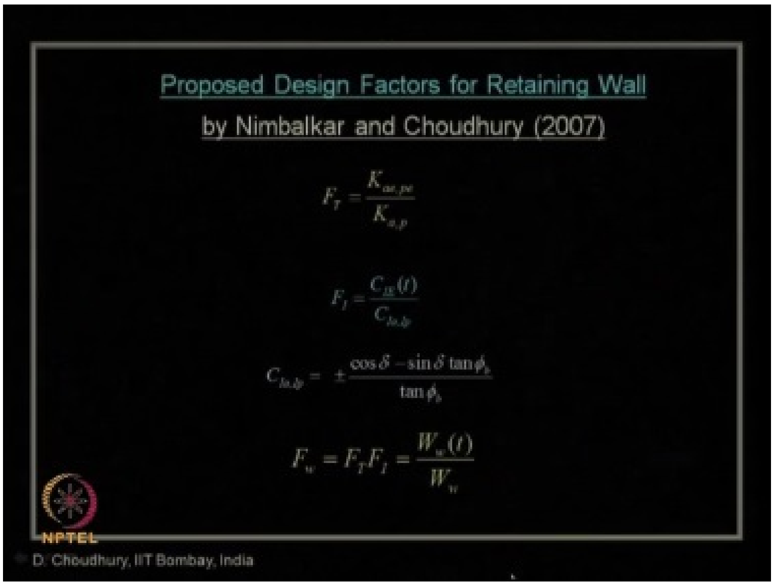

Here also the same combined dynamic factor needs to be obtained.

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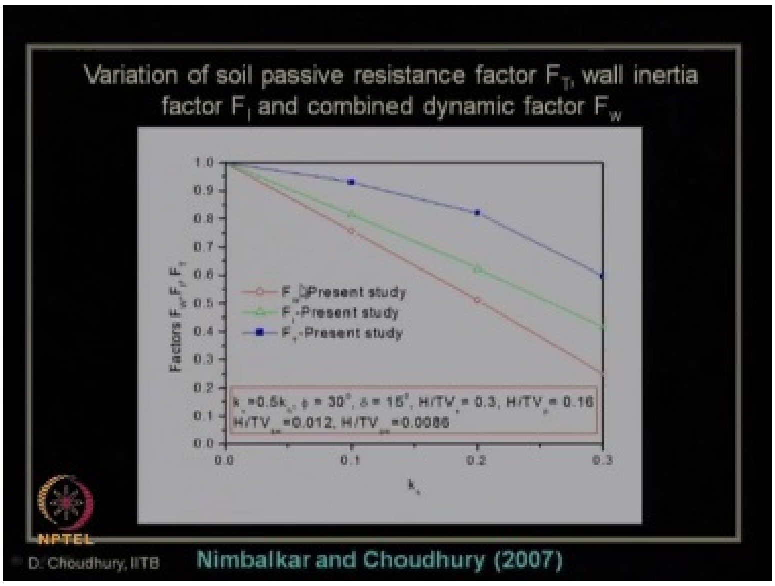

From the proposed design chart like this.

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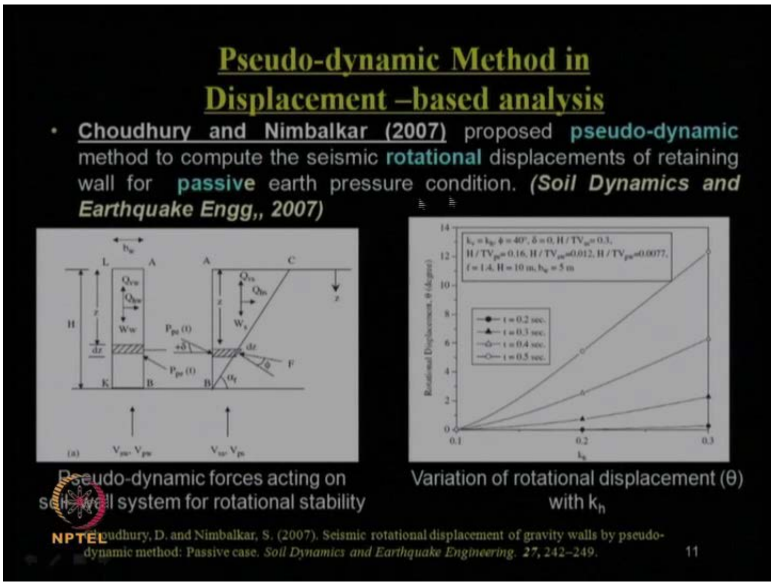

And finally, the section of the wall can be provided. Whereas, this paper – Choudhury and Nimbalkar 2007 – this is also in the journal, Soil Dynamics and Earthquake Engineering. This paper talks about the rotational stability; earlier paper talks about the sliding stability, how to obtain the stable wall section. And, this aspect tells us about how to find out the rotational displacement for a particular wall section with chosen height and width, etcetera. So, once you design the wall with respect to sliding condition, next you need to check for the rotational condition in this manner that, how much rotational displacement you are getting for a particular value of earthquake excitation or input value; how much rotational displacement is occurring. If that value of rotational displacement is more than the acceptable limit or permissible limit given by the designers or the owner of the property; or, as a designer, if you decide that, I will not allow the displacement more than this; that is, performance-based design if you want to carry out; in that case, you can restrict the design section displacement in that manner and redesign the section, so that rotational stability is also ensured under that magnitude of earthquake input motion.

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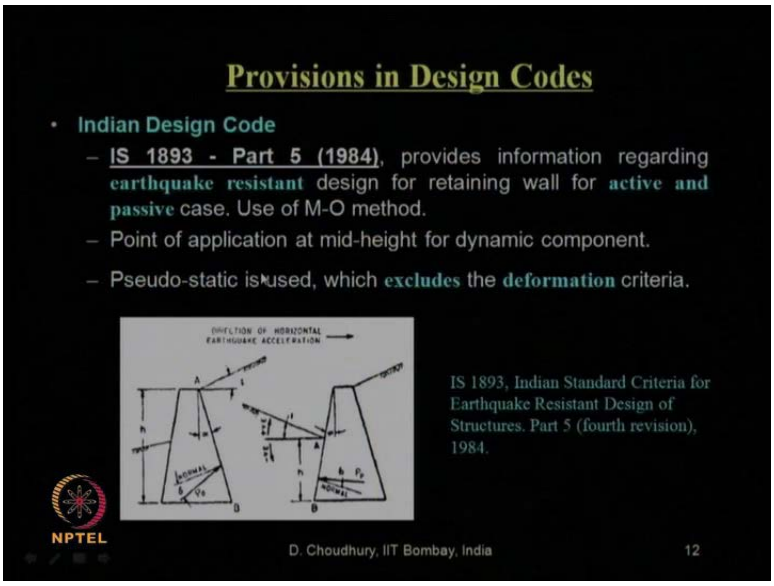

Then, we had also discussed in our previous lecture, what are the recommendations given by various design codes. In that connection, we have mentioned that, our Indian design code IS 1893, though the part 1 has been revised in 2002, that is, the latest version so far; but, for retaining wall design, its version is part 5 of 1984 is still the latest one, where it suggest the use of pseudo-static analysis of age old method of Mononobe-Okabe theory. So, there is no scope for using the deformation criteria or displacement-based criteria. Also, the point of application is a thumb rule. It says that, it acts at the mid-height only for the dynamic component; the static component acts at one-third from the base; but, the extra dynamic component – it acts at the mid height. That is what the code suggests.

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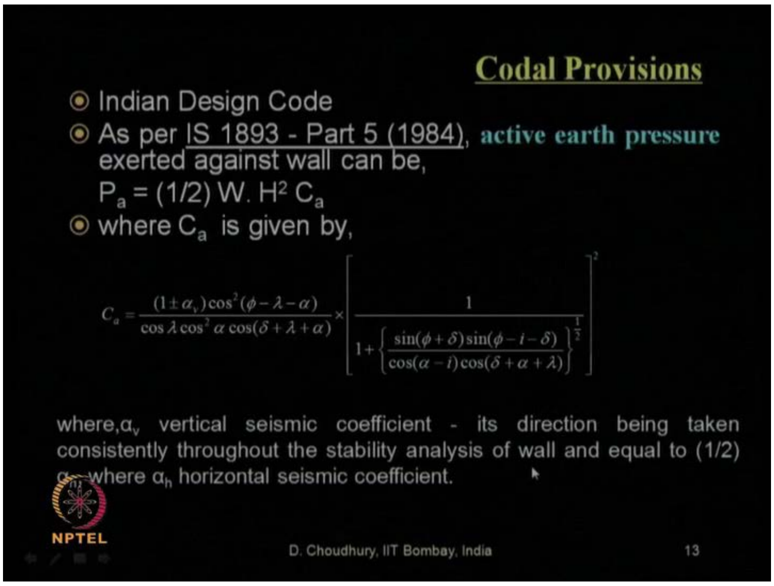

And, this is the equation, how to obtain this active earth pressure coefficient under seismic condition using this Mononobe-Okabe approach as per the IS code. So, this formula is nothing but Mononobe-Okabe's equation of seismic active earth pressure coefficient.

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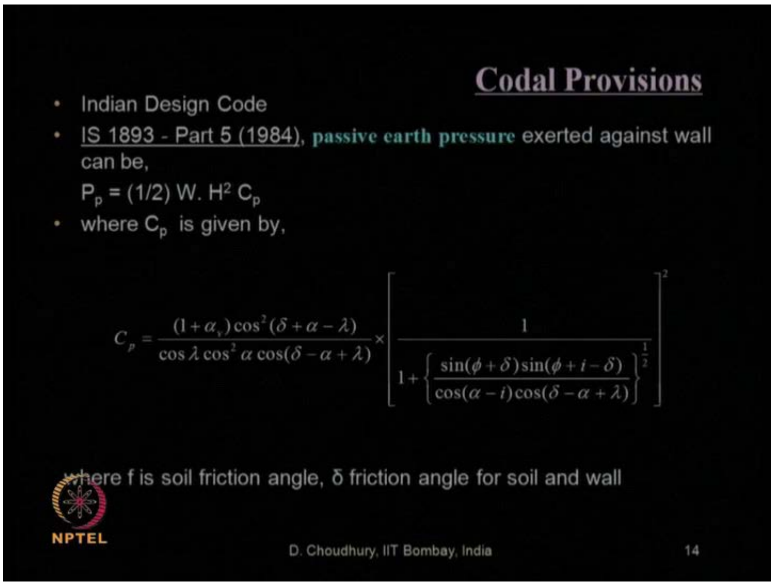

Similarly, for seismic passive earth pressure coefficient using pseudo-static approach as proposed by Mononobe-Okabe, the same formula is given in the IS code also.

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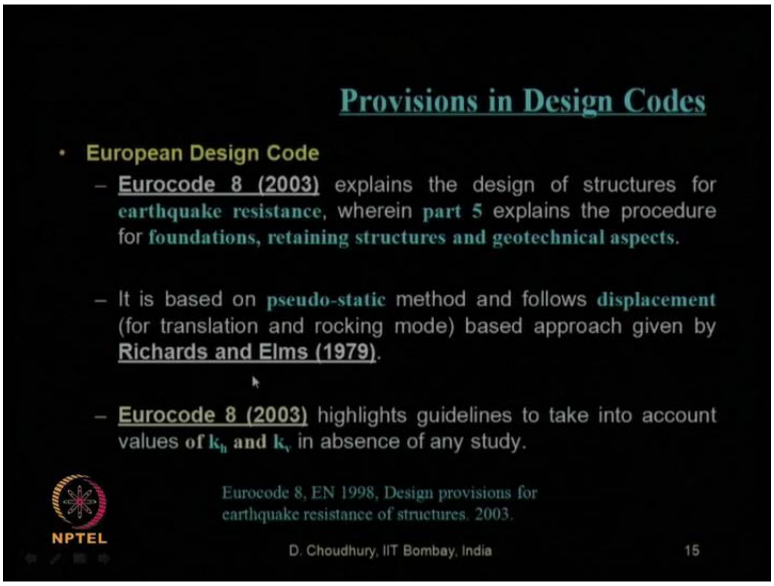

Whereas, we have seen that, European design code, that is, Eurocode 8 of 2003 version; that discussed more about the displacement-based criteria though the analysis proposed the pseudo-static approach. But, they considered the performance-based design criteria or the displacement criteria in terms of translational as well as rocking mode as given by Richard and Elms.

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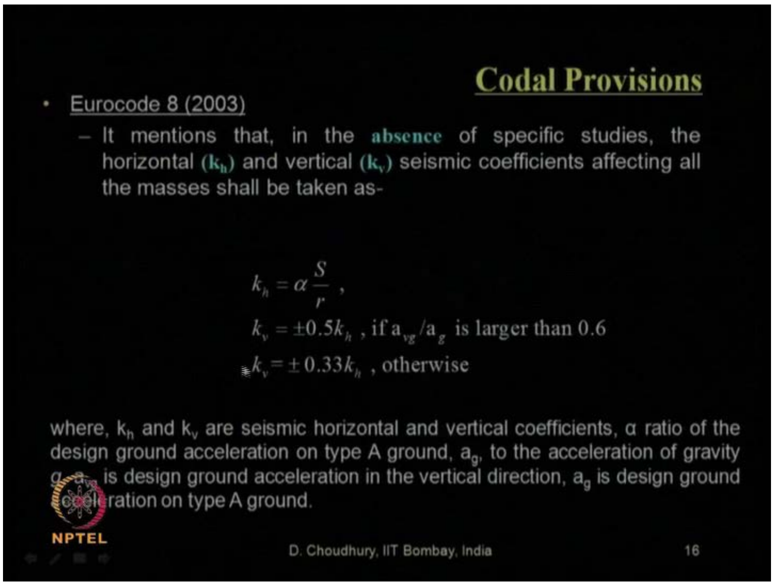

So, how to calculate the values of k h and k v – those input parameters of pseudo-static seismic horizontal and vertical acceleration as per Eurocode? That is, when you do not have the specific study or specific site response analysis or the seismic hazard analysis, if it is not available for the site; in that case, you can use this formula given in the code to obtain the design value of k h and k v.

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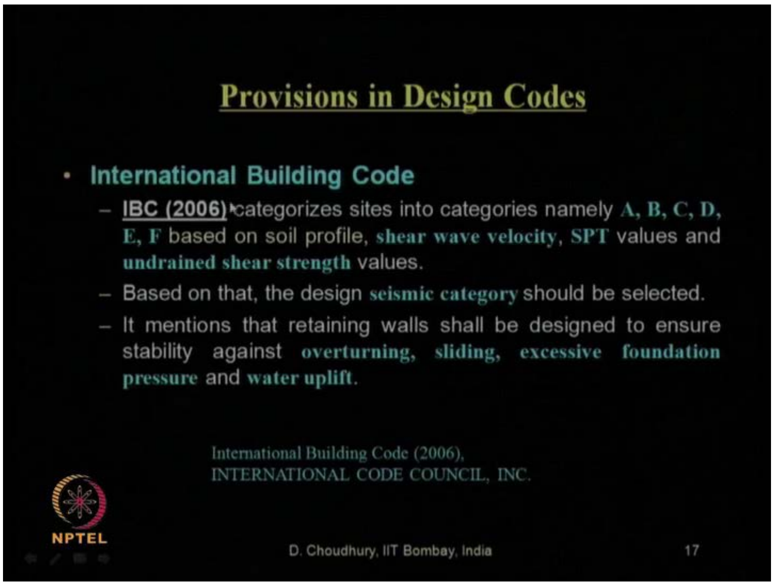

Then, international building code IBC 2006 – they classified the soil or the site into six major categories that we have already discussed when we discussed about the NEHRP code of U.S. based on shear wave velocity, SPT value, undrained shear strength, etcetera. So, based on that, based on different seismic category, it is asked to design the retaining wall, which will be stable with respect to overturning, sliding, excessive foundation pressure and water uplift. So, with that, we had completed our lecture on the previous day.

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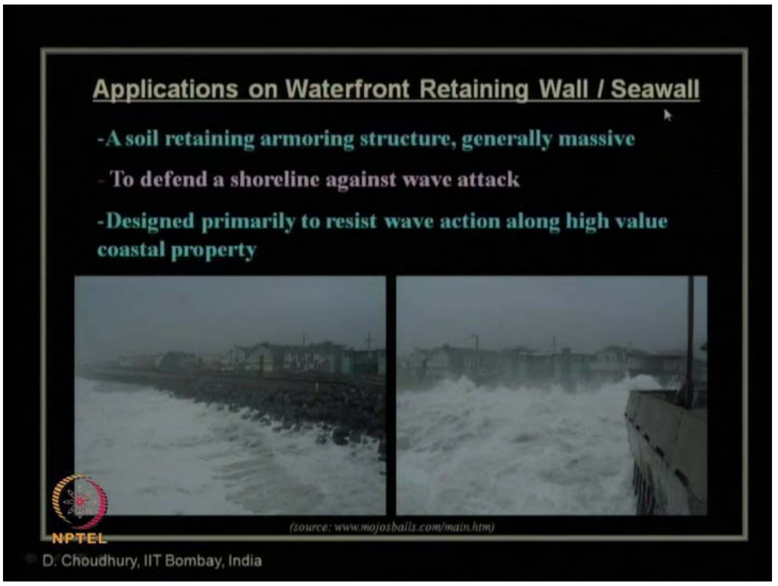

Now, today, we will start with another subtopic, that is, seismic design of waterfront retaining wall. What is waterfront retaining wall? Let us see through this picture. Another name of the waterfront retaining wall is… Let us look at the slide. Waterfront retaining wall is also called seawall. Why it is called seawall? Because you can see in these two pictures, this wall is nothing but to protect the shore from the sea; that is, whenever this water waves from the sea comes and hits the shore, that place, where we provide the retaining wall or the retaining structures, which withstand this water pressure to protect the shore, that is known as nothing but waterfront retaining wall or seawall; the same thing, but different names. So, it is nothing but a soil retaining armoring structure. Generally, it is massive in nature. To defend a shoreline against the wave attack, it is used; designed primarily to resist wave action along high value of the costal property. So, for example, in India in marine drive, we have in Mumbai, this type of waterfront retaining wall or seawall, which is available.

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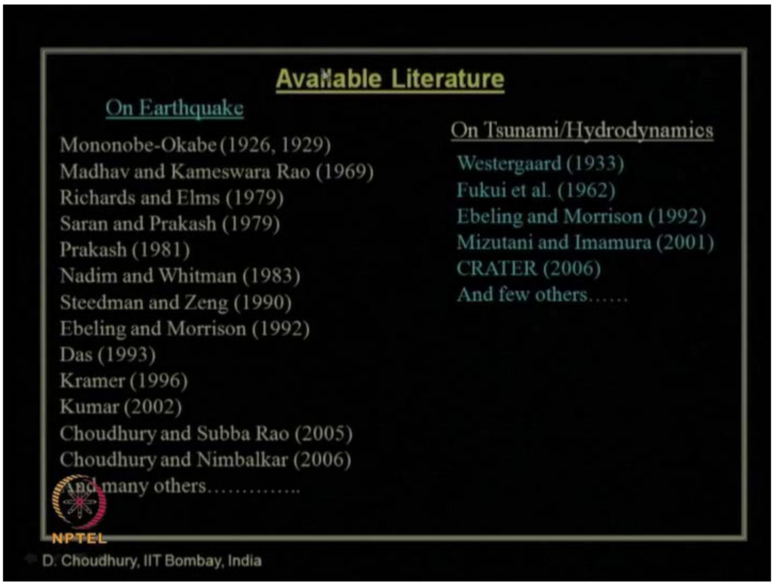

Now, let us look at the research done to take care of different aspects of these design of waterfront retaining wall. Under static condition, the design criterion of this waterfront retaining wall is very well-known. We have to consider not only the earth pressure coming from the soil side, but in addition, we need to consider the wave pressure coming from the water side also. And, remember, in this case, the soil site also will have a water table. So, we need to consider the water pressure from both the sides in addition to that wave pressure coming and hitting on the wall that needs to be considered.

Now, let us move it little further ahead for our course, which is on geotechnical earthquake engineering to address the issue of stability of such waterfront retaining wall or seawall under earthquake condition. Now, let us mention that, when any earthquake comes in the waterbody or seabed, we have already discussed, there is a chance or possibility that tsunami may get generated; it is not always occurring. We have discussed in detail what are the reasons, where it will occur, where it will not occur, etcetera in one of our previous module. So, if during earthquake, the tsunami occurs; and, that waterfront retaining wall is subjected to both the tsunami wave pressure in addition to the static water pressure. Remember, tsunami wave pressure is much higher than the still water level pressure right. So, when we need to design those waterfront retaining wall or seawall, which is protecting the shore from the sea, we need to design against; in addition to that, tsunami wave pressure as well as under earthquake condition, what are the extra dynamic pressure coming from the earth side and what does the extra dynamic pressure or hydrodynamic pressure coming from the waterside or waveside?

Now, let us look at the literature, which are available so far. On earthquake, we have already discussed these names. I have already referred to you in one of our previous lecture that, there are several researchers, who did work extensively for design of retaining wall under earthquake condition. Now, separately, there are several other researchers, who did the design of waterfront retaining wall or sea wall under the tsunami or hydrodynamics. But remember, they have not considered the effect of earthquake. So, what they have considered? Only the tsunami wave pressure they have considered and the hydrodynamic pressure they have considered, but not the seismic earth pressure, which is coming from the backfill side or soil side or from the shore side. So, who are the researchers, who did extensive work on this hydrodynamics or tsunami on the retaining wall? Like Westergaard's method is the most pioneering work or most popular method, which was established in 1933 to estimate the hydrodynamic pressure on a retaining wall. Then, later on, Fukui et al in 1962 designed the retaining wall or seawall with respect to tsunami wave pressure.

Then, Ebeling and Morrison in 1992 designed the retaining wall to withstand the tsunami. And, that is the one, which is used by US army corps of engineers so far for design of waterfront retaining wall. This approach is given by Ebeling and Morrison – 1992. Later, Mizutani and Imamura in 2001 – they also proposed, how to design this seawall with respect to tsunami. And, Crater – 2006 – it is the code, which provides in the US. This a US code, which provides the guidelines – how to design the retaining wall under the tsunami cases. And, few other researchers also worked on this. But, what was the limitation of this available literature? Nobody considered the combined effect of this earthquake along with tsunami, as I said.

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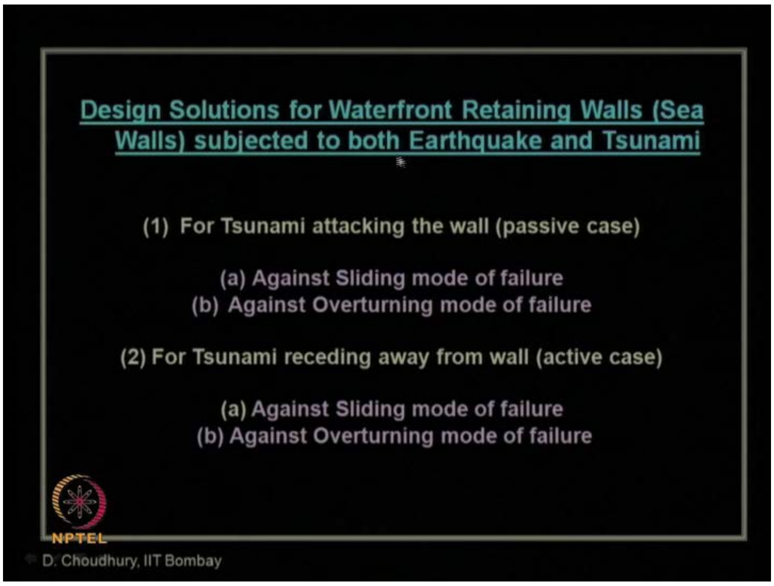

But, people can always mention that this combined effect of this earthquake and tsunami on seawall – it is very very rare. Yes, it is very rare; but what I can state here, from the example of March 2011 Tohoku earthquake in Japan, we already learnt that major damages occurred because of this tsunami. And remember, the major earthquake was followed by several aftershocks. So, earlier, people used to say that, whenever we are designing a seawall or retaining wall, we have to design only for either earthquake or tsunami, because both are extreme conditions; like for extreme condition, we are designing it. So, it should take care of the combined effect.

But, the Japan Tohoku earthquake of 2011 proved it wrong. It said or it showed through several damages that, after the major shock, there can be several other aftershocks, which are also of prominent nature; not of negligible magnitude, but of prominent nature, which are occurring along with the tsunami wave. So, when we are designing this sea wall or waterfront retaining wall, we need to consider the combined effect of this earthquake as well as this tsunami. So, to do that, the research work was done by my second Ph.D. student – Dr Syed Mohammad Ahmad, who is now a faculty lecturer at the University of Manchester in U.K. He did Ph.D. at IIT Bombay. He analyzed this seawall or waterfront retaining wall for two major cases. What are the possible two major cases? First case is when the tsunami is attacking the wall. When let us look at here. Suppose this is our wall and this side is the water. So, tsunami wave is coming from this side; and, this side, we have the shore or the soil side and structures, etcetera over here. So, when tsunami is hitting over here, wall tends to move in this direction towards the soil site; that means, it generates the state of earth pressure, which is similar to passive state. So, that is tsunami hitting the wall we call.

Another case occurs when tsunami comes and then it overtops also; sometimes the wall, because of its height as we know. Then finally, through the weep hole or filters, we know that, finally, the water will go back. So, when the tsunami goes back or recedes back to the sea, because tsunami wave – it comes; then finally, after sometime, it recedes back or goes back to the sea again. So, when tsunami goes back, it tries to drag the wall, because entire water pressure comes from here. So, it is trying to drag the wall towards sea side; that means wall tends to move away from this soil; which is nothing but a state of active earth pressure. So, that is why, you can see in this slide, there are two major cases: one is for tsunami attacking the wall – this is passive case of earth pressure; and, tsunami receding away from the wall or going back to the sea, that is, the creating active state of earth pressure. So, for both the cases, in his Ph.D. thesis work of Dr Syed Ahmad, he worked for stability aspects with respect to sliding as well as overturning in both cases of passive as well as active.

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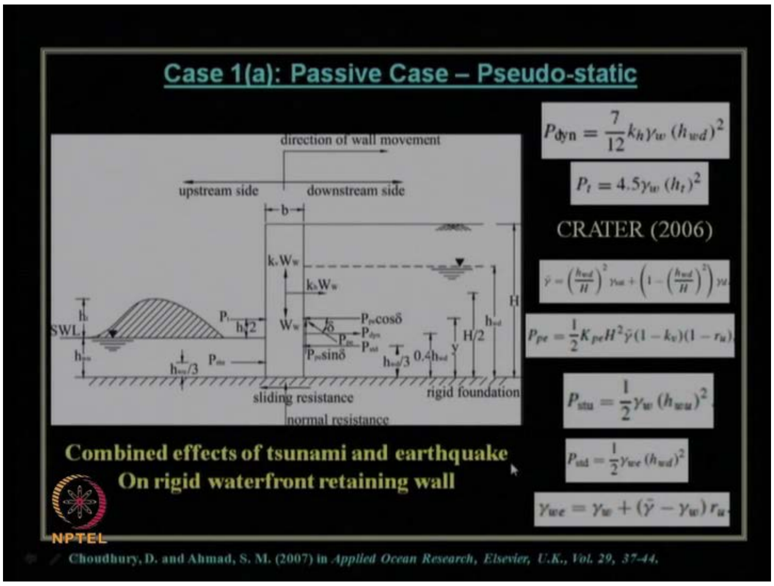

So, let us see, what are the details? First case let us consider that is passive state of case, that is, when tsunami wave is hitting the wall or attacking the wall and considering the pseudo-static method of analysis. So, this is a typical line diagram you can see to consider the combined effect of earthquake and tsunami on this rigid waterfront retaining wall. This is the line diagram of rigid waterfront retaining wall. This is the downstream side we have named here; that is, the shore side, where soil is there; this is the ground level. And, let us take; this is the water table level. So, now… Earlier, all the earth pressure problem we consider for dry soil. Now, we are introducing this water table effect also. So, remember that. So, this height of the water table we have considered as small h w d; h w is the height of water; d means downstream side; and, capital H is the total height of this wall. And, on this side, that is, upstream side or the water side or sea side, this height a h w u is nothing but the still water level. So, SWL is nothing but still water level of the sea. But, when tsunami comes, there will be extra height on top of this still water level. Let us say this is h t, is the height of the tsunami wave.

Now, if we see what is the water pressure acting from this upstream side on the wall; from this static still water, it will be P static; u means upstream; P st means P static upstream. And, where it should act? At a height of h w by 3, because it is a hydrostatic pressure. Now, tsunami pressure P t needs to be calculated. That tsunami pressure is acting on the wall in this direction now, because it is attacking the wall. Let us say it is acting at a height of h t by 2 from this sea water level. Now, direction of wall movement is towards the earth. That is why we said it is passive case. Now, within the wall, the pseudo-static seismic inertia forces will be k v W w; W w is the weight of the wall section. So, this is the vertical inertia force; this is horizontal inertia force k h W.

Now, from the soil side, what we will be having? P p e is the passive earth pressure under earthquake condition at an angle delta with respect to normal. So, we can resolve it into two components of cos delta and sine delta. Now, see here are two components. P std means this height of the water what it provides the water pressure. So, it is a static water pressure, P st in the downstream side, which will act at the one-third of this height h wd. Now, there is another additional pressure remember, which is P dynamic. What is P dynamic? That is nothing but hydrodynamic pressure. This hydrodynamic pressure – look at the direction; it should be in the opposite direction; not… Like hydrostatic pressure acts in this direction, here also hydrostatic pressure acts in this direction; tsunami wave pressure is acting in this direction; but, hydrodynamic pressure is acting in this direction. Why? Because this wave is generating the dynamic pressure –

hydrodynamic pressure and giving on the wall. So, that is why, this P hydrodynamic is acting in this direction acting at a height of this one.

How to estimate that hydrodynamic pressure? We used the equation given by Westergaard. As I said, Westergaard's equation for hydrodynamic pressure is very well-known and commonly used worldwide, which gives 7 by 12 k h times gamma w times h w d whole square. So, h w d is the height of the downstream water table; gamma w is the unit weight of the water; k h is the horizontal seismic acceleration coefficient. So, that gives you the estimate of this P dynamic. What is P t? P t is nothing but tsunami wave force. So, tsunami wave force is given by this expression – 4.5 times gamma w h t whole square. This is given by this code crater of 2006. So, this is P t and this is h t. Based on the tsunami wave height, you can calculate what is coming tsunami wave force.

Now, how to calculate this seismic passive earth pressure P p e? In this case, P p e – as we have mentioned, we are using pseudo-static approach. The similar equation of Mononobe-Okabe is used here; which gives… Half K p e is seismic passive earth pressure coefficient; h square… Remember, instead of gamma now, we have defined another parameter gamma bar. Why gamma bar? Because now it is no longer a dry soil. You need to consider the effect, is soil unit weight, because of this dry zone as well as this saturated zone. And, k v is the vertical coefficient of seismic acceleration. And, what is r u? r u is pore pressure ratio.

Now, what is this gamma bar? Gamma bar is calculated in this fashion. h w d by H whole square gamma saturated plus 1 minus h w d by H whole square whole to the times gamma d. Gamma d is nothing but dry unit weight of the soil above this water table level. So, this way, the average gamma of this entire soil layer is calculated based on what is the height of the water table. So, this is also available in the book of Kramer, 1996. This equation you can see there, because it is a pseudo-static approach. So, this expression is available. How much is the P static in upstream side? This hydrostatic pressure in the upstream side – that will be half gamma w, is the water unit weight h wu whole square.

And, what is the hydrostatic pressure in the downstream side? Half gamma w e h w d whole square. Here why it is not gamma w, but gamma w e? Because see here – what is gamma w e? Is the effective water pressure, which is coming here; gamma w plus…

Already you have taken this gamma bar – effective unit weight of the soil in this earth pressure calculation. So, you are removing this portion gamma bar minus gamma w times r u from this; otherwise, you will be taking water pressure two times. Got it? Why it is deducted? This is the reason why this gamma w e is used on the downstream side. So, this was published by us, that is, Choudhury and Ahmad. As I said, this is the Ph.D. thesis work of my second Ph.D. student, Dr Syed Mohammad Ahmad. Choudhury and Ahmad, 2007 paper in the journal – Applied Ocean Research, which is an Elsevier publication volume 29, page number 37 to 44. You can get all these details in this paper.

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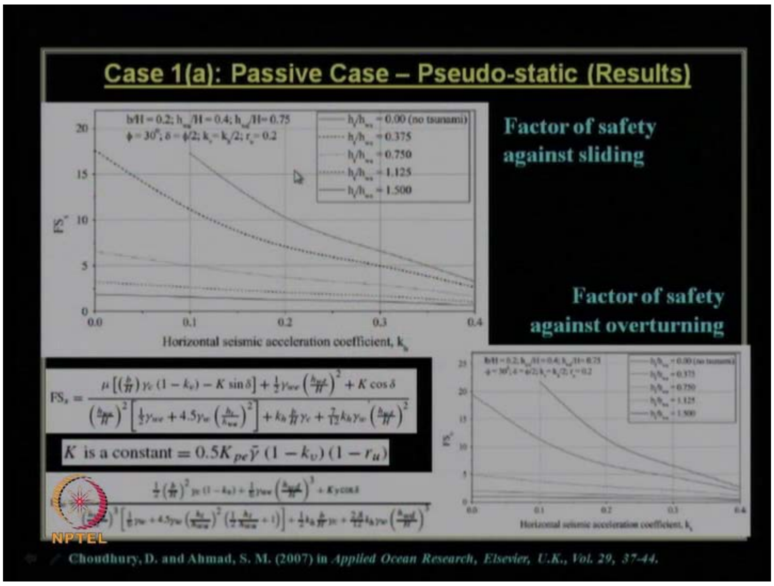

Now, let us see the results – what we have done after using the simple limit equilibrium approach for all the forces involved with respect to sliding as well as overturning mode of failure. Then, the results with respect to the factor of safety against sliding and factor of safety against overturning are shown. So, this figure shows factor of safety value against sliding FS s with respect to various values of horizontal seismic acceleration coefficient k h for a given value of b by H ratio, h w by H ratio, h w d by H ratio, phi value, delta value and k v value and r u value; for this different curve shows for different height of tsunami wave – H t with respect to h w u. Can you see? h w u is nothing but still water height. When it is 0; that means, no tsunami; but, when it is 1.5 means the tsunami wave height is on top of the still water level 1.5 times of the height of that still water level. So, for different values, we got the different results; which is also available as we have proposed in a closed form solution or a complete equation form; which can be used by practicing engineers or practical design engineers directly. How? This factor of safety with respect to sliding is calculated mu times; mu is nothing but coefficient of friction between the base of the wall material and the soil material against direct sliding.

And, b by H ratio is the property of the wall section, which you want to design. So, typically, the b by H ratio for design we considered at site between about 0.2 to 0.4. That is the typical range for a rigid retaining wall. So, using those values, you can do an iterative study as long as you get a safe factor of safety of more than 1.5 even under this earthquake and tsunami combined condition. So, this k v is the vertical seismic acceleration coefficient. This k is a constant, which we have defined as this expression; that 0.5 times K p e gamma bar 1 minus k v times 1 minus r u. How r u is known? For a particular soil with known water table and saturation condition, you know the pore pressure ratio. So, that pore pressure ratio and given input value; you can calculate gamma bar. As I have already mentioned, knowing the water table level, K p e, you can use the Mononobe-Okabe or only other pseudo-static seismic passive earth pressure coefficients.

And, these all others are non-dimensional term of ratio; that h w d by H, that is, height of the water table with respect to total height of the wall; h t by h u is the tsunami wave height over the static still water table level; gamma w is nothing but unit weight of water; and, gamma w e is nothing but equivalent unit weight of the water. So, using this 1… And, this gamma c is nothing but unit weight of the concrete, which you are using for the waterfront retaining wall or the sea wall. So, using all these values or combinations, one can easily compute the factor of safety against sliding; or, they can refer to these design charts and get for a particular tsunami wave height, what should be the factor of safety for this chosen section. Remember, this is for a typical chosen data set of value. If you change the value, these values obviously, is going to change as given by this equation.

Similarly, for the overturning mode of failure also, the factor of safety is represented. And, can you observe one thing? Earlier, we know, for the static condition, for the no tsunami condition, that factor of safety against sliding is decreasing drastically as the seismicity increases or seismic acceleration increases. Now, in addition to that, you can see, as the tsunami wave height increases, the factor of safety is decreasing also drastically. Can you see that? So, this is for 1.5. Next one is 1.125; this is for 0.75, like that. So, as the tsunami wave height is increasing, factor of safety for a given earthquake excitation is also decreasing. So, we need to take care of this combined effect of tsunami wave height action as well as the seismic acceleration value for the design with respect to sliding and overturning stability of the wall. So, this gives the details this journal paper – Applied Ocean Research as I already said.

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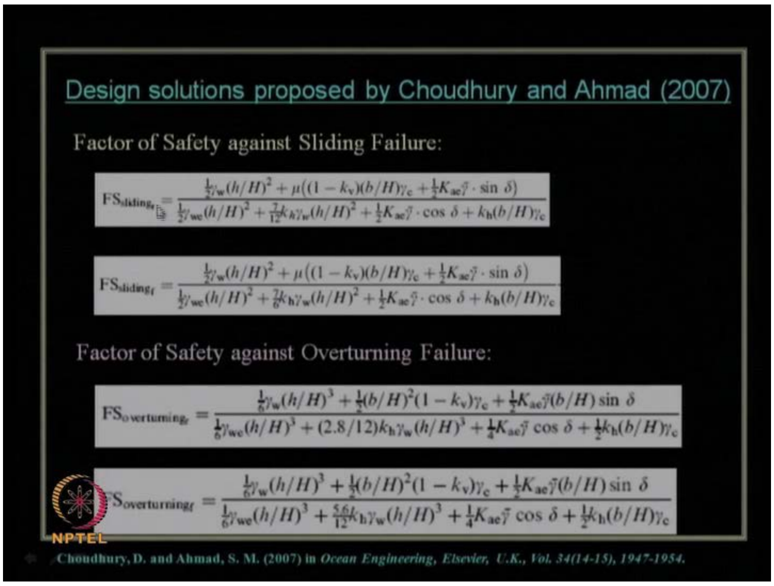

Now, coming to some more results for factor of safety against sliding, this is for the other case of active state. Earlier, we talked about the passive state. Now, we are proposing the design of wall section, when the tsunami wave is going back to sea or the receding back. So, for active state, the design solution were proposed by this paper – journal paper – Choudhury and Ahmad – 2007 in the journal, Ocean Engineering. It is also Elsevier publication. This is the volume and page number. You can see, here also, we have proposed the combined closed form solution, which directly the designers can use in practice in a simplified manner; that is, factor of safety in this case with respect to sliding, there are two equations you can see: one is factor of safety sliding with suffix r and another is f. What are these? These two cases – why it is arising?

Let me explain through this basic figure. Now, we are talking about the active state; that is, wall of movement is in this direction; that is, tsunami wave is going back. Now, depending on the soil permeability and how you have designed the weep hole for this wall, it will depend that, whether the water which came from the tsunami and now going back will be restrained on this back side of the wall along with soil or it will be of free flow of water. That is why, there will be possibility of arising two cases in active state. In passive state, there were only one case. But, in active state, for sliding itself, you have two combinations or two conditions based on the permeability of the backfill material. If it is good permeable material, that is, say sandy material, good sand with high value of permeability with very good designed weep holes or filter drains, etcetera, you should use the… Which equation you should use? In that case, you should use factor of safety against sliding in free-flow condition. But if the permeability is not good and the filter design or weep holes are not designed properly; in that case, the soil is going to retain back some water. So, that should be used for restrain water condition. So, factor of safety sliding with respect to restrain. So, for both the cases, the closed-form equations are given. Similarly, for the overturning mode of failure also, there will be two cases of factor of safety: one is with respect to restrain water condition; another is free-flow water condition. You can get the details in this journal paper.

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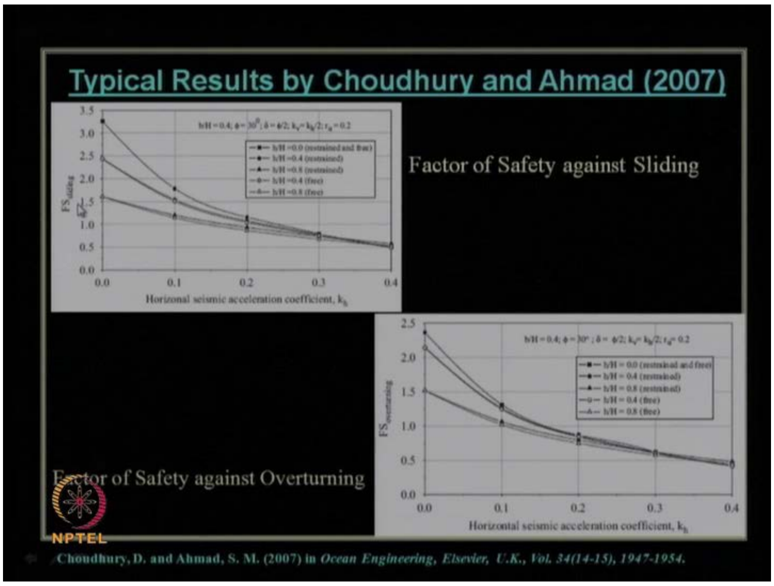

These are few results: factor of safety against sliding and factor of safety against overturning for both restrained and free water case. You can see here, what are the changes in the factor of safety values and with respect to the horizontal seismic acceleration. In that case, remember, you need not to consider tsunami wave height, because now, there is no reason to consider that. Tsunami wave, when it is attacking, then you are concerned about the height of the tsunami; when it is going back, there is no height available. So, that is why, it is not with respect to tsunami wave height, but it is with respect to what height of the water it is standing on the downstream side or in the soil side; based on that, you should do the design.

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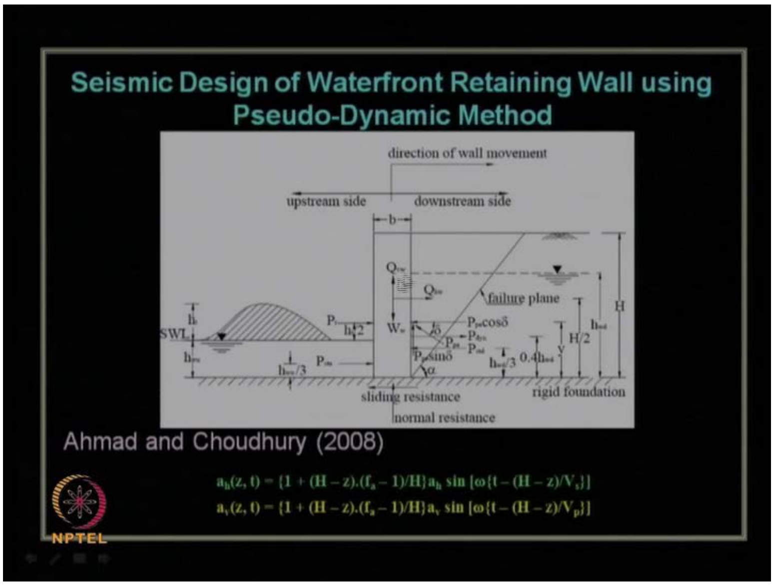

Now, let us come to the application of this pseudo-dynamic method. So far, we applied the pseudo-static method for calculation of this wall inertia, soil inertia, etcetera. Now, we are applying the pseudo-dynamic approach, which we have developed as we have mentioned in earlier lecture. And, these are the basic equations of seismic acceleration in horizontal and vertical direction, which we have already mentioned. So, first, let us talk about the passive case again; that is, when tsunami is hitting the wall. That designed criteria is given by Ahmad and Choudhury in 2008.

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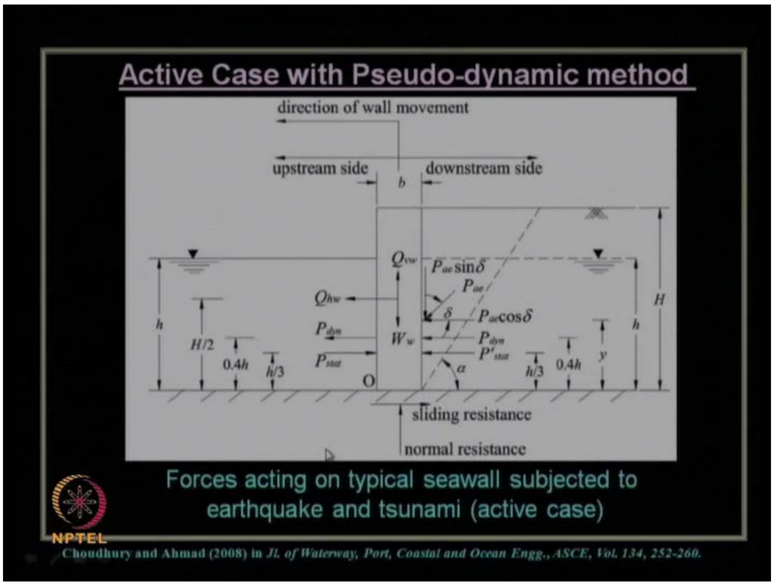

This is the journal paper; you can get the details; Journal of Waterway, Port, Coastal and Ocean Engineering published by ASCE, U.S.A. This is the volume number and page number – ASCE publication journal publication and for active state of earth pressure. This is for the active state; that is, wall movement is in this direction considering both earthquake and tsunami; that means, tsunami wave is receding back or going back to the sea side.

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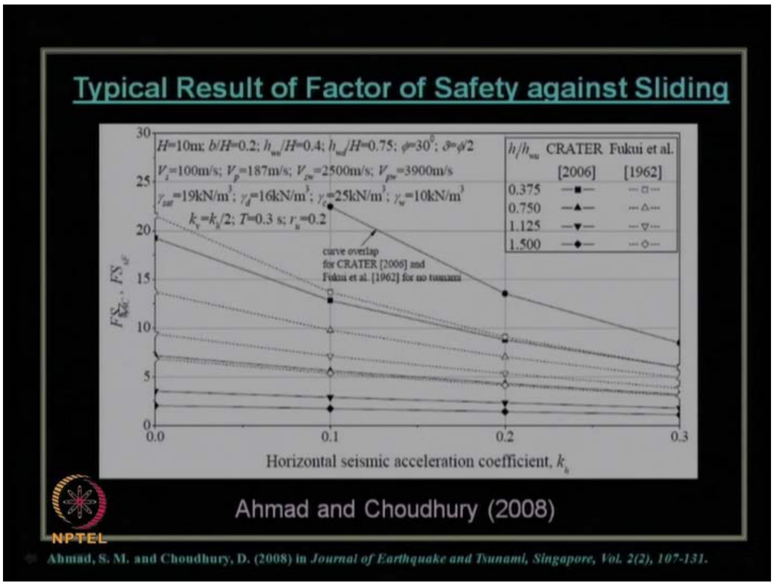

And, these are the results you can see over here; that is, considering the Crater value and Fukui's value; that is, one is Japanese method to calculate the tsunami wave force; another is American way to calculate the tsunami wave force. Correspondingly, the factor of safety for different cases is reported over here considering Crater's approach and Fukui's approach.

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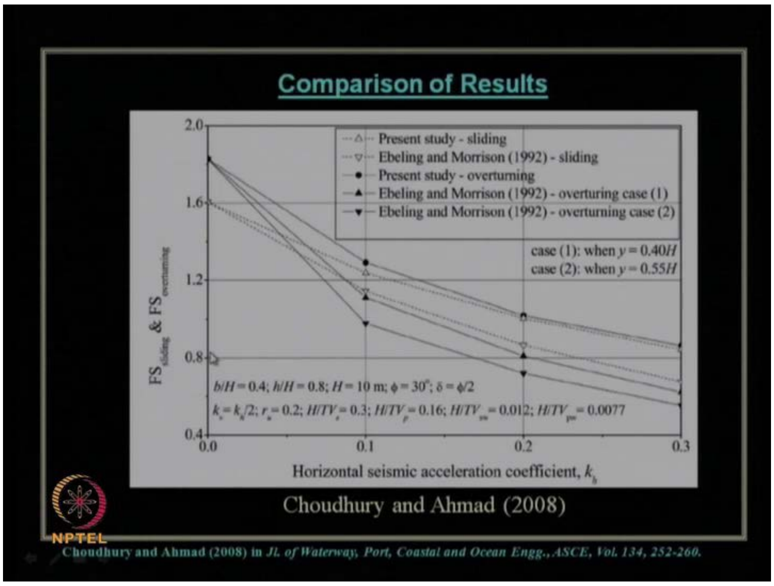

Now, if we want to compare the results… For this factor of safety again, sliding and overturning with the available results of the pseudo-static values of Ebeling and Morrison in 1992, which I said was used by US army corps of engineers for design of waterfront retaining wall. You can see the pseudo-static values are showing not the critical design, but the pseudo-dynamic method – the present study shows the more critical design; that means, it is giving more precise and more safer design as far as the pseudo-dynamic method is concerned compared to the pseudo-static method for waterfront retaining wall design. Now, with that, we have shown how to design the waterfront retaining wall or seawall to take care of both the effects of tsunami as well as earthquake.

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Now, let us move to the seismic design of reinforced soil-wall. This is our next subtopic, that is, seismic design of reinforced soil-wall. We all are aware about the use of this reinforced soil-wall. It is a very practical common examples, which are nowadays people use. In other courses, I am sure you have the basic knowledge about what is reinforced soil-wall; where it is used; where there is a space constraint, etcetera. To get the higher stability with the verticality of the wall, etcetera, we use the reinforced soil-wall. So, geo-synthetic reinforced soil-wall – there are several applications worldwide. This picture is from Japan. You can see a Japan rail is moving, which is getting supported by using this geo-synthetic reinforced soil-wall.

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And, in terms of our course of this geotechnical earthquake engineering, let us see how this geo-synthetic reinforced soil-wall performs during an earthquake. This is a case study as reported by Professor Tatsuoka in 2010 in his paper that, remember, when the Kobe earthquake in 1995 – that is most one of the major damaging earthquake in Japan, which occurred earlier before the Tohoku earthquake of 2011; you see, soon after the earthquake, all the buildings, structures, everything got collapsed; but, this geo-synthetic reinforced soil-wall that survived. So, the wall survived even that high magnitude of Kobe earthquake motion; which is used for the rapid transit they call, that is, rail system in Japan in Kobe region. That shows directly that the geo-synthetics reinforced soil-wall – they perform even better than the conventional gravity type retaining wall under earthquake condition. So, we need to know how to design this reinforced retaining wall.

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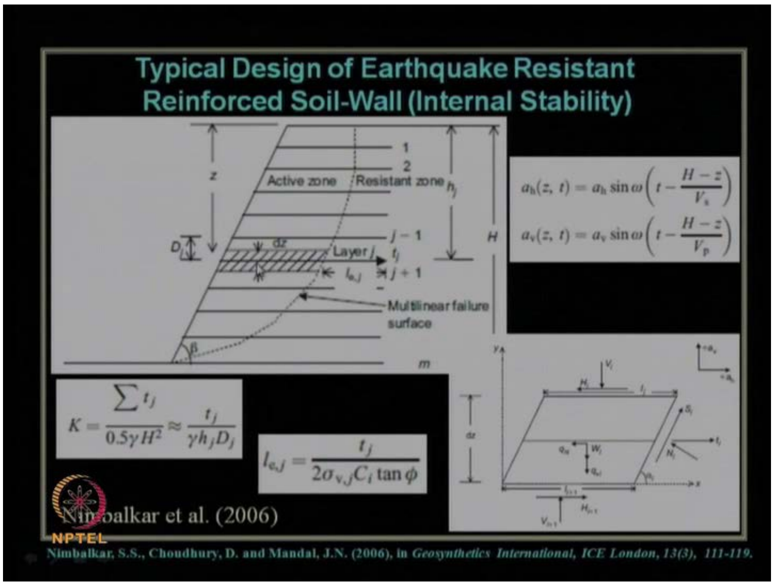

There are two basic design criteria for reinforced soil-wall: one is called internal stability criteria; another is called external stability criteria. So, among these two stability criteria, let us first discuss internal stability. What is internal stability? It is meant by that stability of reinforced soil-wall section, so that the reinforcement strength is proper enough to take care of the strength or the stresses coming due to that vertical height, etcetera. So, whatever reinforcement you are providing, that should have sufficient tensile strength, so that it can withstand extra large height, which we are providing by using this reinforced soil-wall. That should not break. Also, the anchor ridge of the… or pullout of this reinforcement layers should be ensured. These all aspects comes under the criteria of internal stability of the reinforced soil-wall. And, what comes under external stability? External stability is nothing but the entire reinforced soil-wall zone should not fail against sliding, overturning – those are coming under external stability.

Now, when we talk about internal stability, this is again I am referring to a part of the Ph.D. thesis work of my first Ph.D. student Dr Sanjay Nimbalkar. This is the paper details you can see – Nimbalkar Choudhury and Mandal in the journal Geosynthetics International published by Institute of Civil Engineers, London in U.K. This is the volume and page number in 2006. So, what we have done in the analysis, this reinforced soil zone – there are several layers of reinforcement as you can see. A small infinitesimal element has been chosen at a depth of z with the thickness of layer as dz. How this layer was chosen? So that, within the layer at the center of layer, there is one reinforcement. That way the element has been chosen.

Why? Because it should not be chosen at the interface of the reinforcement and soil; otherwise, there will be shear stresses in this way or additional stresses coming from this tensile strength of the reinforcement. So, if you look at this infinitesimal small element, which is shown over here in this picture, what are the forces acting? These are shear and normal force on both the sides of the slice – this horizontal slice; and, this t j is nothing but tensile strength of this reinforcement, which is at the center of this small infinitesimal layer. And, what is W i over here? This is nothing but weight of this small infinitesimal section. And, this q h i and q v i are the seismic inertia force in horizontal and vertical direction. Here we have used pseudo-dynamic approach, because as I said, Dr Nimbalkar developed pseudo-dynamic approach and he used that method for all his analyses. So, these are the input seismic acceleration in horizontal and vertical directions – the sinusoidal seismic acceleration.

Finally, the results are obtained in terms of this parameter K. What is this parameter K? This is a non-dimensional parameter, which represents the ratio of the strength of the tensile strength of the reinforcement layer expressed in terms of with ratio of gamma is the unit of the soil; h j is the thickness of that particular layer at which layer you are using that tensile strength required; and, D j is the interval between the two reinforcement layers. So, using that, it is non-dimensional. Once if you get the results in terms of K for different input values of H or a v or in terms of k h or k v in terms of coefficient, you know how much reinforcement needs to be provided, so that it can withstand that much seismicity.

Another requirement is there for internal stability, which is in terms of length of reinforcement. How much of this length? One is strength – how much is the strength of the material you are providing? That is, which material you have to use, which is having that much strength, which can withstand that particular design value, which you are getting from this proposed method. Another aspect is how much length of that reinforcement you are going to provide, so that there is no pullout failure or any other internal failure of the reinforcement. So, how to find it out? This is the expression. This is the effective length of that j th layer l e j with respect to t j; that is, the strength of the reinforcement tensile strength divided by 2 sigma v j C i times tan phi.

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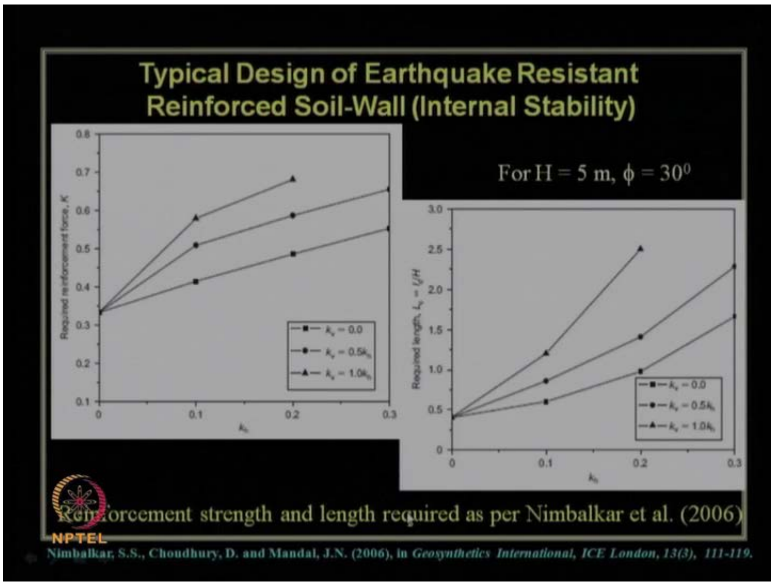

These are the few typical results you can see, how practicing engineers or design engineers can use our proposed design chart. Let us look at here. Suppose for the seismic zone you know, what is the k h value – input k h value; let us say it is 0.2 g. And, let us say, the k v value for 0.5 times of k h – we need to do the design. So, where we should go? We should go to this point. Now, you get the value of this required reinforcement force in terms of that non-dimensional value of k. So, you get the value; say it is about 0.6; little less than 0.6 say 0.57, 0.58. Using that, now, what we can do? Let us go back. If you know this value, you know the gamma of the soil; you know the h j and D j; how much you are going to provide the interval of the… or thickness or interval of the individual strips of the reinforcement. Using that, you can get what should be tensile strength of the reinforcement, which you want to provide.

Another thing – required length, that L c – how you need to find out? Same for k h, say 0.2 we want to design; with k v, say 0.5 H. So, this point we have to go. Length is coming say close to about 1.5 times of this. So, it is expressed again in the non-dimensional form of L c by H. Let us go back here. So, this is our… this value of L. It is expressed in terms of this non dimensional height of ratio of H. So, you know how much height you are going to design or achieve. So, knowing the value from this figure, say 1.5 and H, you get to know your l e value over here. So, once you know the L e, if you go here, t j already you have calculated; l e you know. So, provide that much length.

If you want to check whether you are getting that much coefficient with respect to direct sliding or not, that is another cross check. If you are not getting that direct sliding, you change the value over here. So, this is the coefficient of friction with respect to direct sliding of the material. So, if you have a different material, you will have different input values of C i tan phi, which is nothing but the interface friction between the reinforcement and the soil. This is how the design can be made using our proposed approach as given in this paper. So, these are few comparisons.

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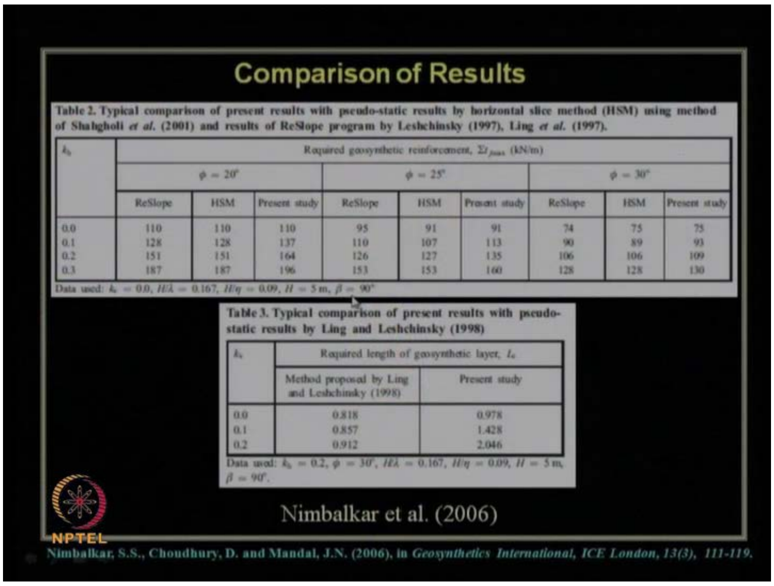

Now, we have also validated our results with the earlier available results. But, earlier available results all were using pseudo-static approach. And, ours was pseudo-dynamic approach. So, this table shows the comparison with respect to the work done by Shahgholi et al in 2001. This is the paper published in Geotechnique journal. And, this is the work done by Leshchinsky in 1997 and Ling et al in 1997. These are paper published in ASCE journal of Geotechnical and Geo-environmental Engineering. You can see the comparison for different values of input values of k h. What is the required reinforcement strength is required? That is, in terms of kilo Newton meter, the tensile strength.

Our present study for different values of soil phi value – 20 degree, 25 degree, 30 degree, you can see, how much our present study gives. In most of the cases, our present study at higher values of k h are giving that more critical value of the design. Can you see that?

That is, little higher tensile strength is proposed as per the pseudo-dynamic method is concerned, where you have a scope to consider all the dynamic factors like frequency of excitation, time duration; then, shear wave velocity, primary wave velocity, even soil amplification also, etcetera. And, we have compared our results as you can see over here with respect to Ling and Leshchinsky value of 1998 paper with respect to the how much required length is necessary in terms of that non-dimensional value. Remember that, L c is nothing but ratio of that l e by H. So, for different values of k v and for input value of k h equals to 0.2, our present study suggests that more length of reinforcement is necessary for better stability of the wall. So, the details can be obtained in this paper.

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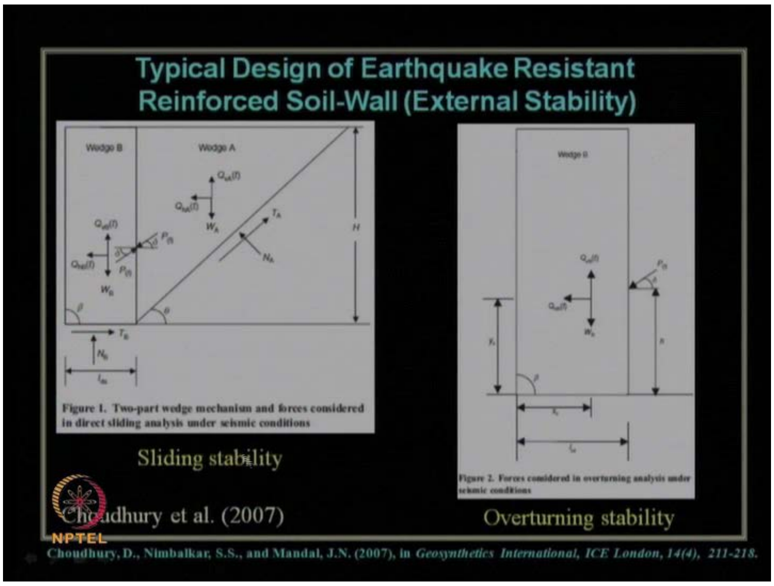

Now, coming to the external stability of this reinforced soil-wall, we now know how to design in terms of internal stability. When we talk about the external stability of this reinforced soil-wall, let us see over here; as I said, there are two cases: sliding stability and overturning stability. The details of this analysis can be available in the journal paper by Choudhury et al 2007, that is, Choudhury Nimbalkar and Mandal 2007 published in the journal Geosynthetics International published by Institute of Civil Engineers, London, U.K. This is the volume number and page number. You can see, in our analysis, we considered a two-wedge failure mechanism; that is, there will a facing material over here; and, this entire wedge A and wedge B is the entire zone of reinforcement – soil reinforcement. So, these are the seismic inertia forces in wedge A, which is triangular in nature and wedge B, which is rectangular in nature.

And, considering the pseudo-dynamic approach, we calculated the stability in terms of sliding for this picture. Again, the stability of the reinforced soil-wall was considered in terms of overturning also considering this effective pressure coming from, active earth pressure coming from this soil (( )) wedge A; and then, acting on the wedge B when it is about to deep about this point, rotating about this point.

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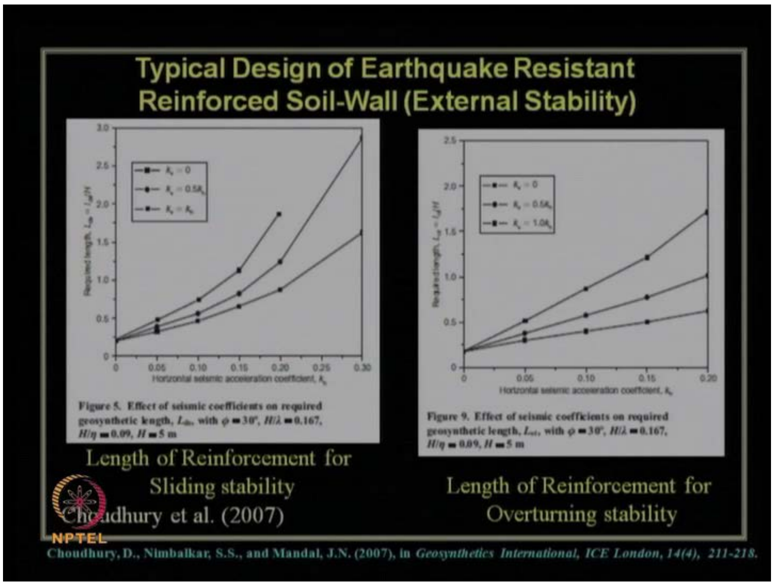

What are the recommendations or the design values of the results or design charts? These are the design charts, which gives us what is the requirement length of the reinforcement. So, remember, in this case of external stability, what we are getting? We are getting, how much length of the reinforcement to be provided for stability with respect to this sliding and overturning. Earlier, we get in terms of internal stability, the strength of the reinforcement and length of the reinforcement in terms of pull out failure. Now, we are getting in terms of sliding and overturning failure. So, again, designers can use this design chart, come to a particular value of k h; and, corresponding k v get the required length – how much is required in terms of sliding failure stability. This is the minimum length required, so that sliding failure will not occur.

Similarly, this is the minimum length required, so that overturning failure is not occurred. So, how much… Finally, when we are designing the reinforce retaining wall to withstand earthquake, what should be our final recommendation? The strength of the reinforcement material we have to ensure from the value of that t j – that proposed design chart of the seismic reinforcement strength from internal stability criteria; and, how much length of the reinforcement to be provided should be considered from three criteria, that is, pull out criteria in terms of internal stability, then sliding failure criteria of external stability and overturning failure criteria of external stability. So, among these three, whichever gives us the maximum value or highest value of the reinforcement length to be provided; that has to be adopted at the practice or at the design at the site.

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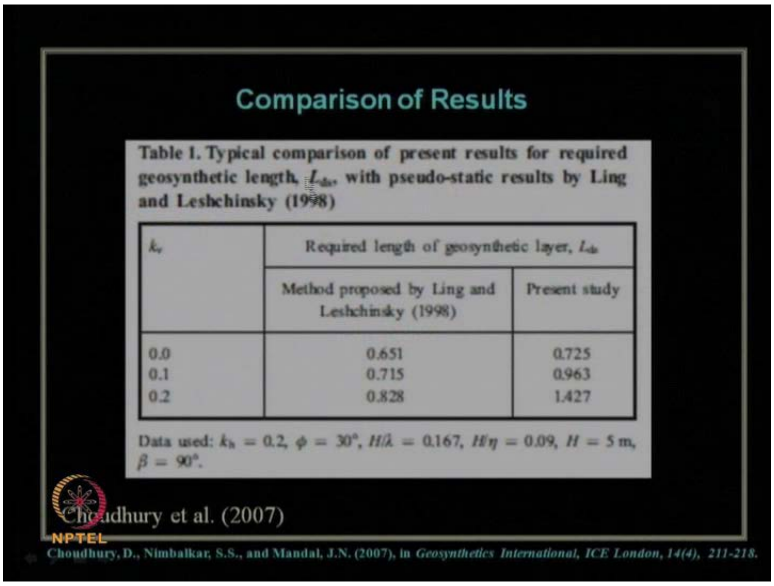

k_v	Required length of geosynthetic layer, L_{ds}	
	Method proposed by Ling and Leshchinsky (1998)	Present study
0.0	0.651	0.725
0.1	0.715	0.963
0.2	0.828	1.427

So, this shows the comparison of the result of this geosynthetic reinforcement length required in terms of direct sliding. So, L ds shows the length required in terms of direct sliding; of course, it is again represented in terms of non-dimensional parameter for different values of k v and k h with respect to Ling and Leshchinsky’s paper of 1998, which is published in ASCE journal. But, this is for pseudo-static analysis. And, our present study here mentions the analysis used by pseudo-dynamic approach, which gives the critical design for the practice to implement, which is available in this journal paper. So, with this, we have come to the end of today’s lecture. We will continue further in our next lecture.