Geosynthetics and Reinforced Soil Structures
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Lecture - 13
Seismic Loads and Internal Stability Analysis of Reinforced Soil Retaining Walls

Good morning students, let us continue our discussion from the previous lecture wherein we have seen how to calculate the length of the reinforced block based on the different modes of external stability failures.

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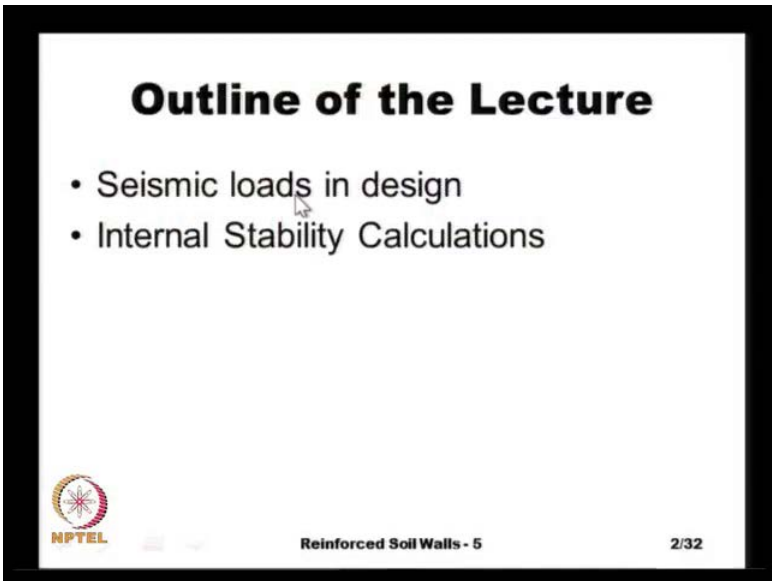

And in this lecture, let us continue our discussions by considering the forces, because of the earth quakes that is the seismic loads, and they also look at the internal stability calculations.

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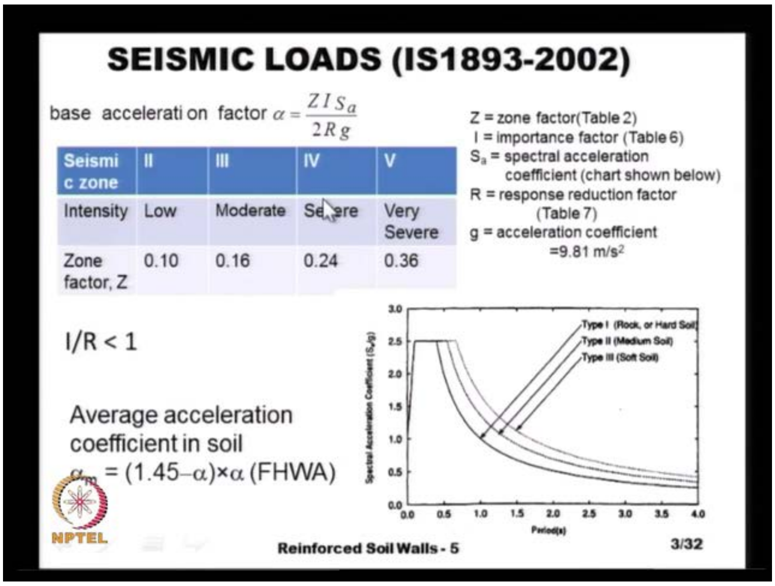

For the purpose of considering the cyclic loads, we have this IS code IS 1893, the revised version released in 2002. And according to IS 1893, the entire country is divided into into four zones, zone 2, zone 3, zone 4 and zone 5 depending on the intensity of the expected earthquakes. The zone 2, being the least seismically active and then zone 5 is the most active zones in a very severe earthquakes could be expected. For example, our Chennai city is in the moderate earthquake zone that is, in zone 3.

And the IS 1893 has given a procedure to calculate the base excitation factors through this formula, alpha is Z I S a by g divided by 2 R. Where, Z is the zone factor wherein, the zone factor is directly taken from this table corresponding to different zones, zone 2, 3, 4 and 5, I is the importance factor that we will see a bit later on and S a by g is given through this chart. Basically, this is the spectral acceleration coefficient that depends on the expected duration of the earthquake.

And then the type of subsoil, whether we have rock or hard soil, medium soil and soft soil and so on and then capital R is the response reduction factor and the g is the acceleration coefficient. And once we estimate this base acceleration factor, we can calculate the actual acceleration coefficient that is active in the soil through this formula, that was proposed by a federal highway administration as a alpha m is 1.45 minus alpha times alpha and now, let us look at the different factors in this equation.

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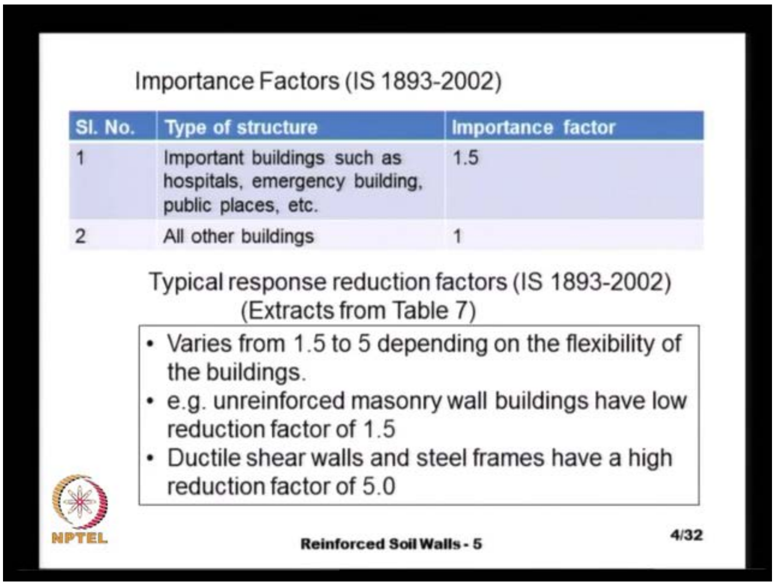

The importance factor is once again given in this IS code and this is given with respect to the structural applications. For example, depending on the type of the structure, whether it is an important building or any public place like hospital building or emergency buildings or a school building like that. And these structures, they will have an importance factor 1.5 and all other buildings will have an importance factor of 1 and we can probably extrapolate this at the soil structures.

See if our retaining wall is going to be part of a very important national highway, we can ascend in a importance factor of 1.5 whereas, for rural roads or the run important structures, we can give an importance factor of 1. And then there is also another factor called the response reduction factor that is, the capital R in the denominator and brief extract from that code is given here. Depending on the flexibility of the structure, the response reduction factor varies anywhere from 1.5 to 5. For example, for very brittle structures like the unreinforced masonry or shear walls, we have response reduction factor 1.5 whereas, for steel frames and highly ductile shear walls, we have a reduction factor of 5.

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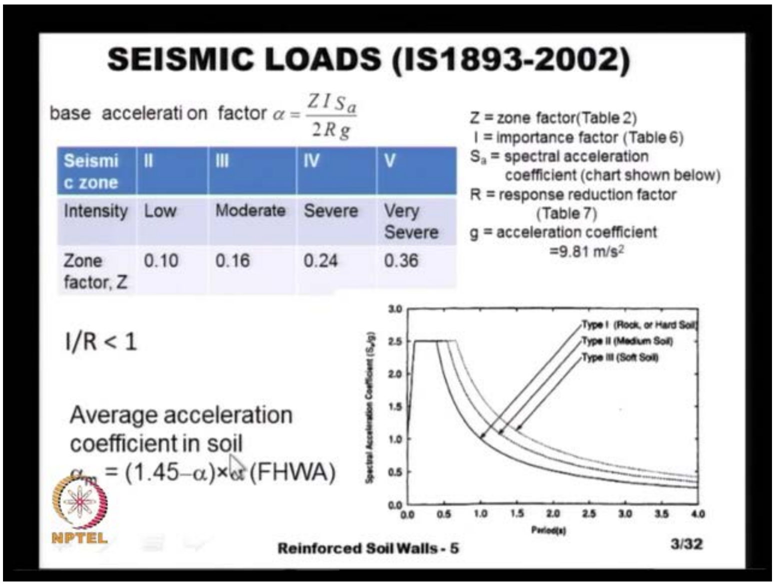

And once we can substitute all the different parameters in this equation and get our base acceleration factor. And then once we have this, we can calculate the average acceleration coefficient within the soil.

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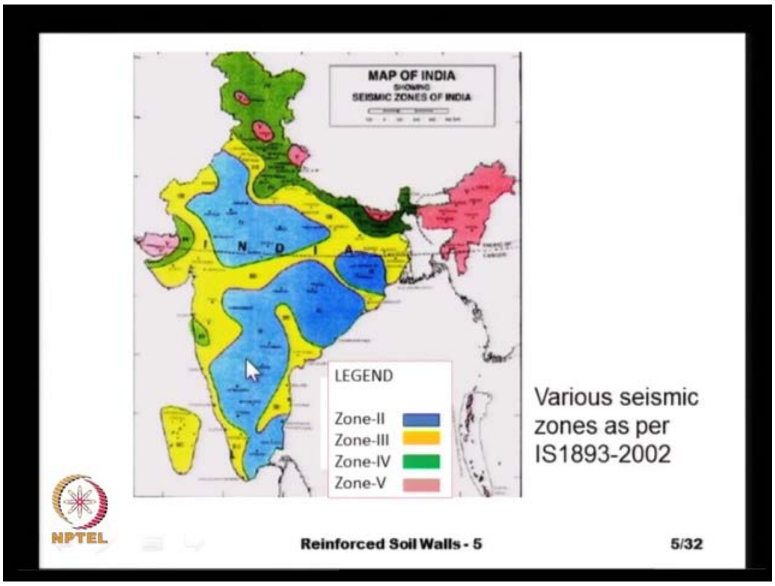

And once we have this coefficient, we can do the other calculations and before we do that, this map shows the different zones, zone 2, 3, 4 and 5. Zone 2 is in blue color, zone 3 is in yellow color, zone 4 is in is in green color and zone 5 is in pink color. For example, Chennai is somewhere here, we are in zone 3 and the Deccan plateau is in zone 2, relatively not very active seismically.

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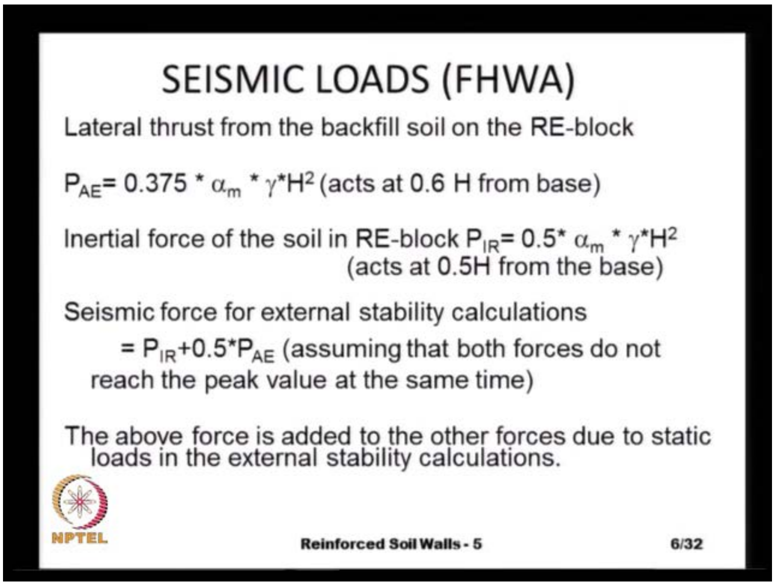

And this the acceleration coefficient, that we calculate can be used for estimating our additional loads on the reinforced soil structures. And once again, this procedure is given by a federal highway administration and there are two components to the seismic loads. One is because of the inertial forces, that are acting within the reinforced soil block and the other one is the lateral thrust, that is exerted by the back fill soil on the reinforced soil fill.

And for the purpose of this analysis, we treat the reinforced soil block as one entity and the back fill soil is another entity. And we can calculate the lateral thrust that is exerted by the back fill on the reinforced soil fill as P A E that is, 0.375 times alpha m times gamma H square. And this particular force acts at a height of 0.6 H from the base and the other component is the inertial force, that is generated because of the movement or the seismic acceleration within the reinforced block., that is calculated as a 0.5 alpha m times gamma H square.

And this particular inertial force acts at a height of 0.5 height from the base and the net seismic force that we need to consider for external stability calculations is P I R, that is the inertial force developed within the reinforced block plus 0.5 times P A E, that is the lateral thrust exerted by the back fill soil. And the reason why, we take only 0.5 P A E is, there are two independent bodies, they reinforce soil fill and the back fill, and both of them may not develop the peak force at the same time, because their natural period of vibration could be different and because of that, the response also could be different. And because of that, we have this method of estimating the total force, because of the seismic loads. And this particular seismic force is added to all the other forces that we have, because of this static loads, the self weight and so on.

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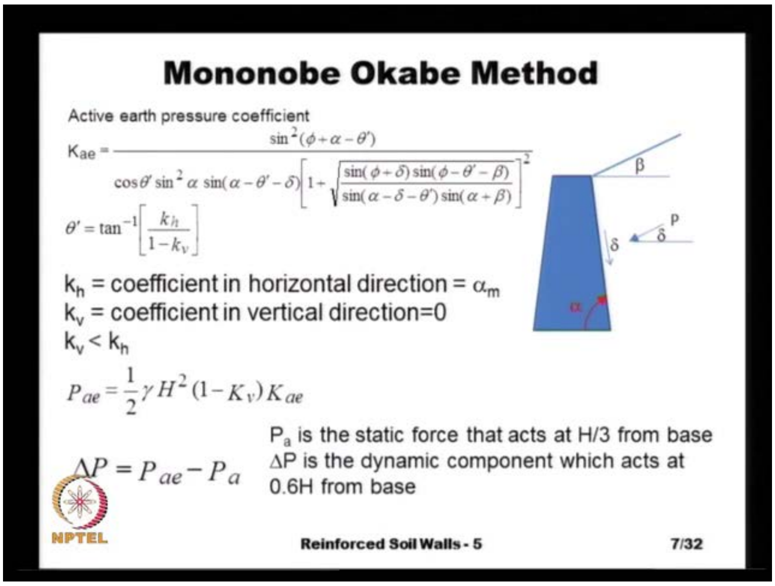

And the previous equation is mostly applicable for retaining walls with horizontal back fills and when we have an inclined back fill, the suggestion is we consider the height of the retaining wall at the back end of the reinforced block, as we have seen in the external stability calculations. And more general method that was proposed by Mononobe and Okabe in 1929 is also applicable for our case wherein our the active earth pressure coefficient K a e, that includes both the affects of the static loads and then the seismic loads is given like this.

And wherein, our theta prime is tan inverse of K h by 1 minus K v and alpha is the inclination of the back surface of the wall and the k h and k v, these are the coefficients of earthquake acceleration in horizontal direction and vertical direction. And the K h, we can take as a alpha m, that is calculated from the earlier calculations and K v we can just simply neglect, because the effect of vertical acceleration is not so significant in the case of soil structures.

And so our net force P a e, that includes both the static component and also the seismic component is calculated like this and the static component P a can be estimated using our regular Coulomb's equation K a. And the delta P is the dynamic load increment, is P a e minus P a and we know that, our P a acts at one third of the base height for self weight and one half of the base height. If we have a rectangular pressure distribution and the delta P is the dynamic component or the seismic component and we assume that, it acts at a relatively higher height that is, 0.6 H from the base.

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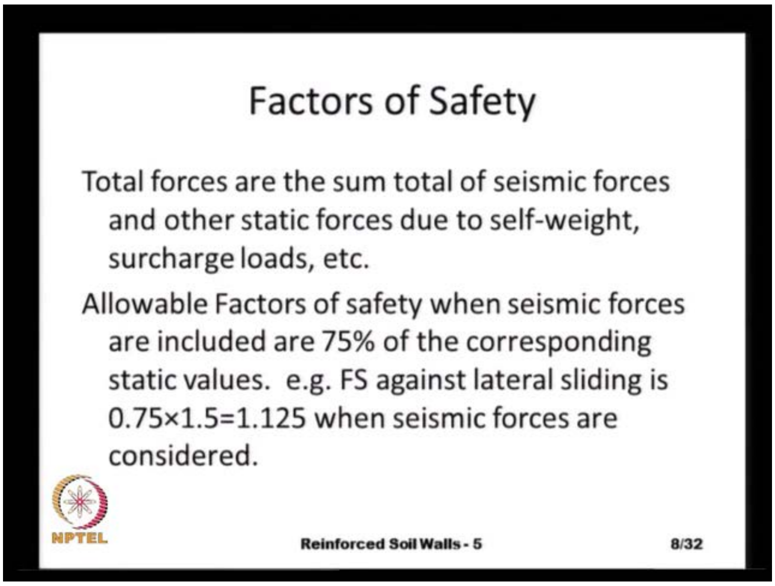

And we can include these seismic forces along with our static forces and compute all our factors of safety against lateral sliding and overturning and then the base pressures and so on. And the allowable factors of safety, when we have this seismic forces can be reduced to 75 percent of the corresponding static values say for example, we required a factor of safety of 1.5 against lateral sliding under the static loads. And when we include the earthquake forces, we can reduce it to 75 percent of 1.5 that is, 1.125 the reason is, the seismic events are extremely rare events. And it is, when we are considering some additional forces, we can have a slightly lesser factor of safety to economize on the designs.

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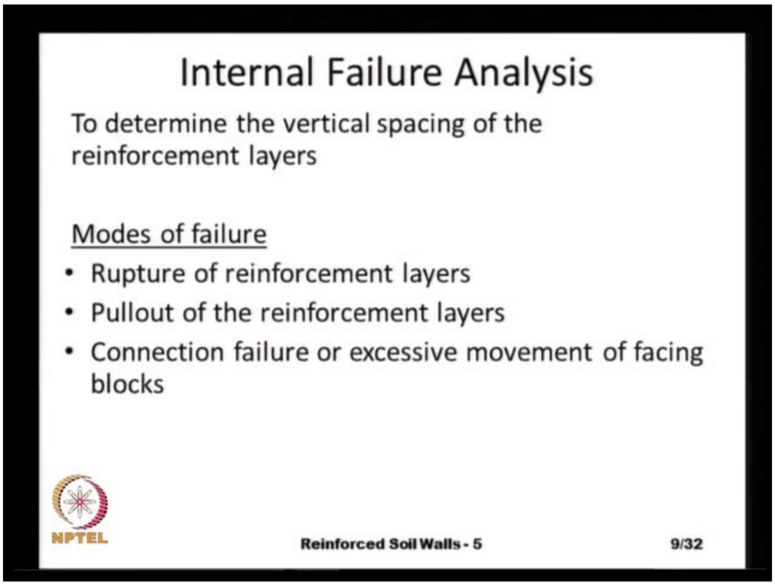

So, once we have done with calculating the length of the reinforcement, we need to determine the vertical spacing of the reinforcement layers and this is done based on three modes of failure and these are called as internal modes of failure. The first one is the rupture of the reinforcement layers that is, when the tensile force that is transferred into the reinforcement layer is more than the allowable strength. The reinforcement may just simply rupture and so we need to have an adequate factor of safety against rupture.

Then, the other mode of failure is the pullout of the reinforcement layers that is, when we do not have adequate length of reinforcement, it might just simply pullout of the soil, because of the forces that are applied in that layer. And the other mode of failure could be the connection failure between the reinforcement and then the front panels or the connection between the different modular blocks and so on.

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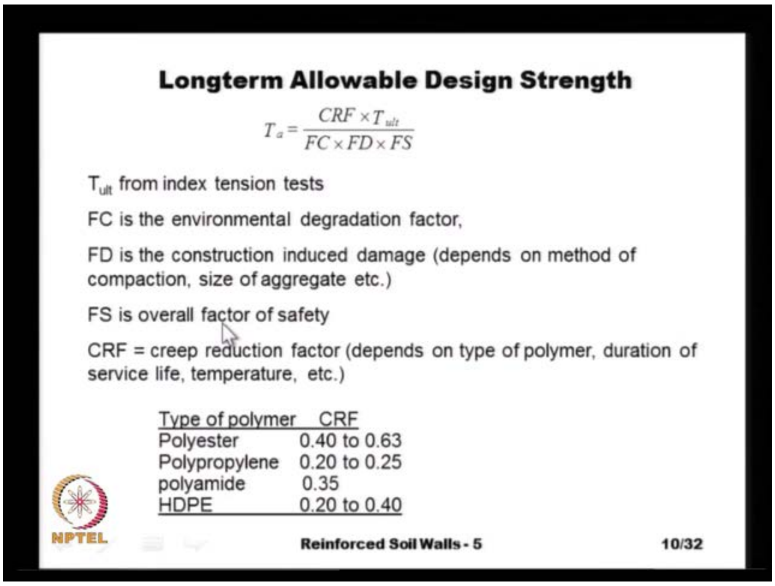

And for all these calculations, we need the long term allowable design strength of the reinforcement material and the T a is written in terms of the index tensile strength that is, the T ultimate, that is obtained from our index test. Wherein, we apply the strain at very high rate 10 to about 20 percent and we reduce that index tensile strength based on number of other factors. The most important one is the creep reduction factor, the creep is the straining of any material under sustained loads and because most of our materials are polymeric in nature, they go on elongating under constant loads that we apply.

And so at the end of our service life, the strains that we have could be much higher than the strains that we have under short term loading. And to account for those factors, we have this creep reduction factor and depending on the type of polymer, our creep reduction factor is different. The polyesters, they are not so susceptible to the creep and because of that, we have a low reduction factors that is, about 0.4 to 0.63. Whereas, high density poly ethylene's, they have as low as about 0.2 creep reduction factor that means, that they are highly sensitive to the creep.

And this particular creep reduction factor is a function of the service life and then the type of polymer and then the operating temperatures that are there at the site. And these are usually site specific and given by the manufacturers, based on their own extensive test data. And the other factors that we have, FC is the environmental degradation factor and FD is the construction induced damage and this depends on the method of compaction that we employ for our construction. And then the size of the aggregate and then the magnitude of the compaction that we achieve, whether we need only light compaction or heavy compaction and so on. And then FS is the overall factor of safety to account for all other unknown factors, either because of manufacturer, because of construction procedures and so on.

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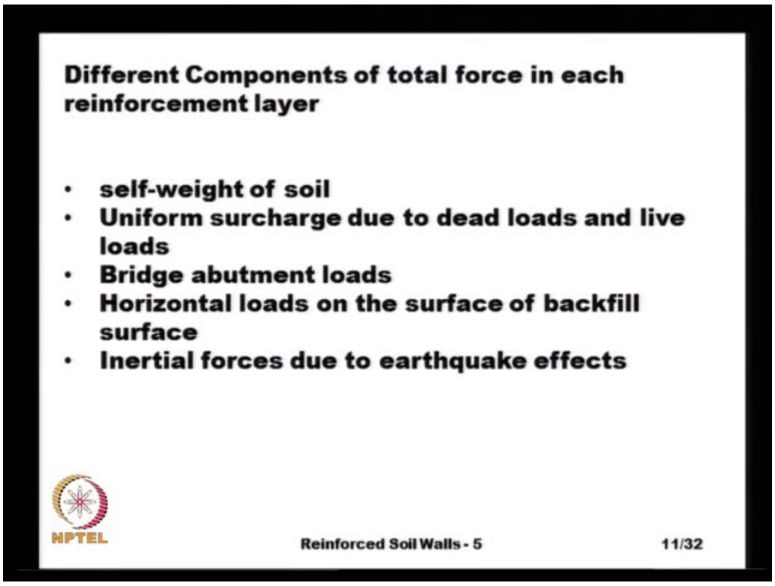

And for the purpose of the internal stability calculations, we need to consider the forces that transferred into a different reinforcement layers, because of the self weight of the soil. And then uniform surcharge loads, because of the dead loads and the live loads and then we may have some loads directly transferred from the bridge abutments, wherever we have a reinforced soil structure that is directly supporting the bridge abutments. And then we could have horizontal loads on the surface of the back fill, because of the interaction forces of the vehicles or the impact force on the guard rails and the other force that we have is the inertial force due to the earthquake effects.

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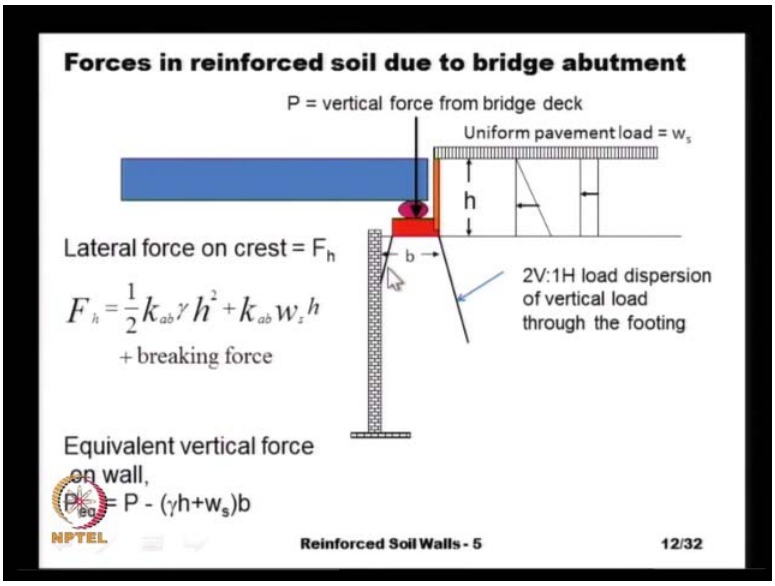

So, typical bridge abutment is something like this, let us say that, this is our reinforced soil retaining wall and we have a bridge abutment that is directly sitting on top of this reinforced fill. And then we have this, this is the bridge deck and this may be transferring some load directly into the reinforced fill through the bearings that we have. Let us say that, the height of this bridge structure is small h and there could be some pavement load that is directly applied on the load surface, either because of the self weight of the pavement structure or because of the live loads that come from the vehicles.

And the lateral thrust on this structure is estimated as one half K a b gamma h square that is, because of the self weight of the soil plus K a b w s that is, because of the surface loads times h and plus the breaking forces. And then other impact forces, that is the lateral force that is assumed to act on the surface of the back fill soil and this vertical force, we treat it in a very simple manner. We assume that, this vertical force is dispersed into the reinforced fill at ratio of a two vertical to one horizontal angle.

And then at different depths, we calculate the pressure as the applied load divided by the base area and on one side, the load is allowed to disperse like this. Whereas, on the front side, after some distance this line is going to intersect our panels, at that time we assume that, it is just simply going vertically. And for simplifying the calculations, we assume that uniform surcharge pressure is acting on the entire back fill surface. And to account for that extra loads, we reduce the vertical force as this P total minus gamma h plus b here, w s times b wherein, b is the front distance that we have.

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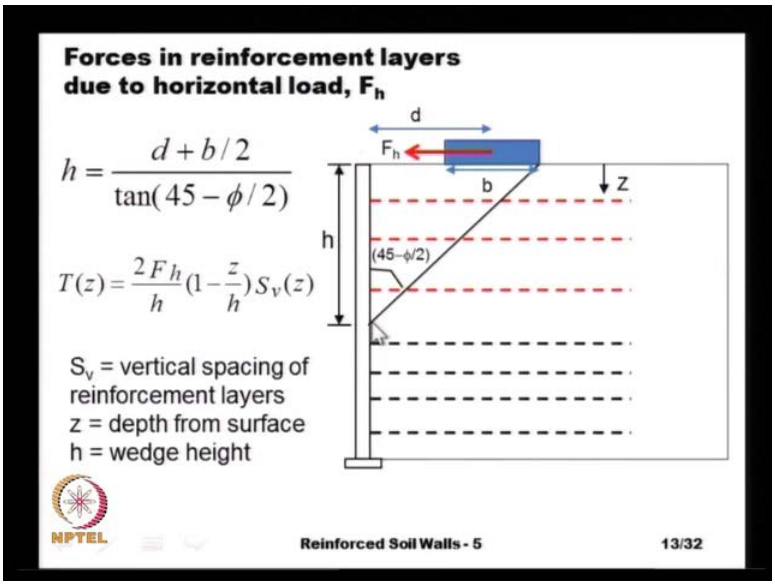

And the horizontal loads that we have on the reinforced soil surface, they are going to exert some lateral force in the reinforcement layers. And we assume that, the only reinforcement layers that are within the active edge, that is drawn from the back of this footing. These reinforcement layers are affected by this horizontal force and the particular reinforcement layer at a depth of z will carry a load as given like this, 2 times the F h that is, the lateral force divided by h.

That is the height of this active edge times 1 minus z by h where, z is the depth of the reinforcement layer from the top surface and S v of z, that is the vertical spacing between different layers. And as we can see, the affect of this lateral load is decreasing in a triangular manner from maximum at the top to 0 at this depth.

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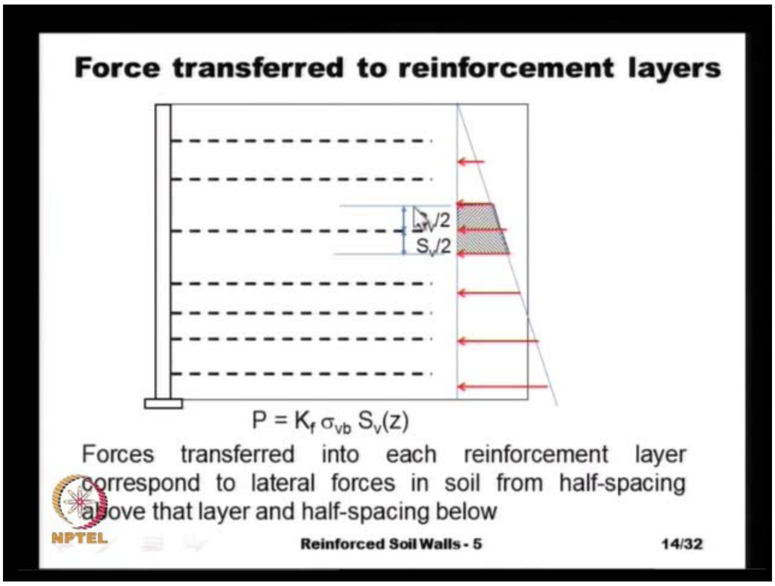

And for the purpose of simplifying the calculations, we assume that, the lateral pressures transferred into each reinforcement layer, only from a height of half the spacing above and another half the spacing below, as illustrated here. And that means that, if we have a pressure of sigma v b at any depth, that multiplied by S v is going to be the contributing pressure or the load and that multiplied by some factor K f. K f is the lateral active earthquake pressure coefficient within the reinforced fill or we should just simply call it as lateral pressure coefficient because it need not be just simply active. And this is a very simple equation, because we assume that, the force that is transferred is from half the spacing above and half the spacing below.

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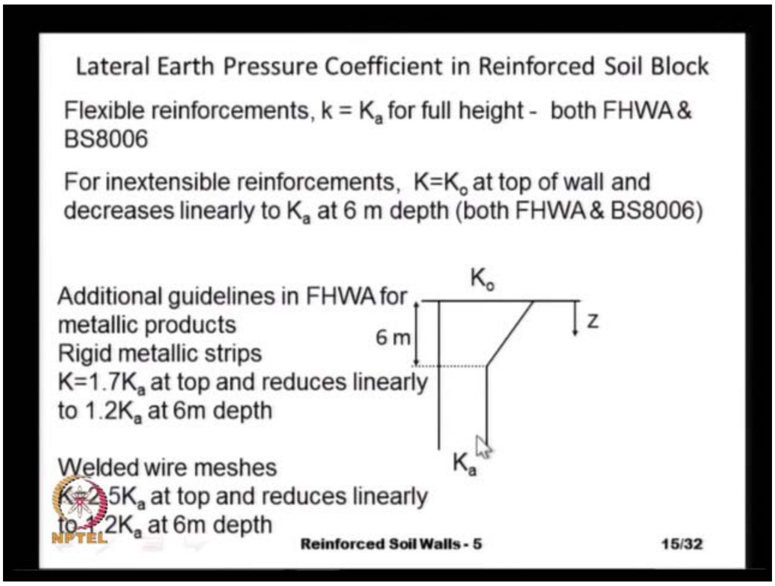

And the earth pressure coefficients that are operating within the reinforced fill, they are given like this. For flexible reinforcements, our k is just simply taken as K a that is, the active earth pressure coefficient over the full height of the wall, both federal highway administration and the BS 8006, they give this recommendation. Whereas, the inextensible reinforcements, they have a more complicated pressure distribution, the k is assumed as K naught at the top, which reduces to K a at a depth of 6 meters as per BS 8006.

Whereas, the federal highway administration, they recommendations are bit more stringent or more conservative. The assumption is that, the k is equal to 1.7 times K a at the top for rigid metallic strips at the top and at 6 meters depth and the pressure reduces to 1.2 times the K a. And the reason that is assigned is, the metallic strips being much stiffer than the soil, they prevent the lateral deformation of the soil. And because of that, the actual earth pressure within the soil does not decrease down to K a and the reason for very high pressures at the top is, most of the structures they are compacted heavily.

And to account for this heavy compaction, we need to take higher lateral earth pressure coefficients and whenever we have these inextensible reinforcement layers, they do not allow the soil to expand laterally during the compaction. Whereas, the polymeric type reinforcements, they allow sufficient deformation so that, our earth pressure reduce to K a. And for the welded wire meshes, the k is much higher, the k is 2.5 times the K a at the top and reducing linearly to 1.2 K a at 6 meters depth and below the 6 meters depth, the k is assumed to be constant at K a.

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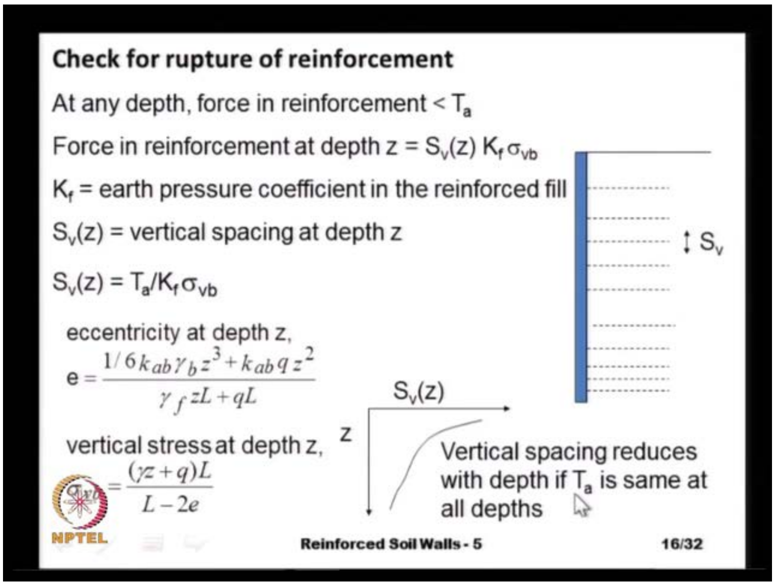

And once we have the pressure distribution, we can equate our the allowable tensile force in the reinforcement T a to the force within the reinforcement layer that is, transferred, because of several reasons, because of the self weight and because of the surcharge loads are because of horizontal loads and because of inertial forces. And if we equate this T a to this quantity, this is just a generic equation and we can calculate the vertical spacing as T a and divided by K f times sigma v b and sigma v b is the may half earth pressure at different depths.

And as you recall, and the may half pressure is calculated as the total load divided by effective width that is, L minus 2 e where, L is our length of the reinforcement layers. And if you plot a graph between the vertical spacing with depth, it is very high at the top surface because the lateral earth pressure are not significant and the spacing goes on reducing like this. And if the T a that is, if we use the same type of reinforcement over the full height of the wall, they reduces or we can choose different type of products.

So that we have higher strength of reinforcement at the bottom and lower strength of reinforcement at the top, so that we can maintain uniform spacing, but for the case where, our tensile strength is the same over the full height of the wall and the vertical spacing reduces like this.

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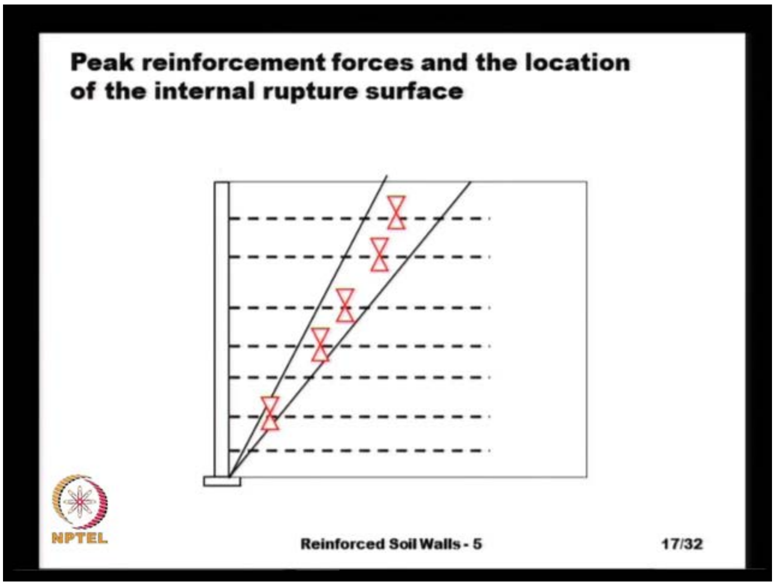

And for the purpose of calculations of the pullout capacity, we need to know where this the wedge stops. And we assume that, we have the Rankine active wedge by drawing a line at the bottom surface at an angle of pi by 4 plus phi by 2. And this particular result is also confirmed by so many other independent testing laboratories.

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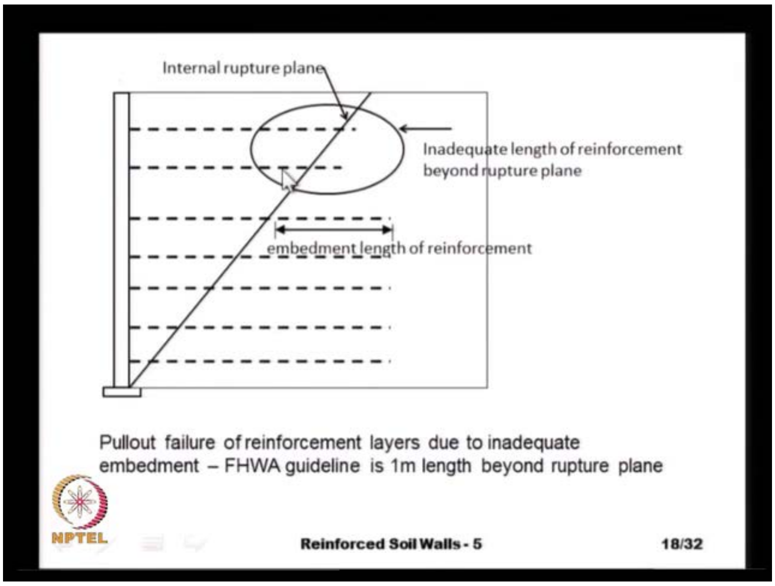

And the purpose for knowing this line is to be able to estimate the embedment length that is, the embedment length is the length of the reinforcement beyond our rupture surface. That is just a simple planar surface for the case of flexible reinforcements and for inextensible type reinforcements, we will see a bit later on. And our federal highway guideline is that, we should have a minimum embedment length of 1 meter beyond our rupture surface.

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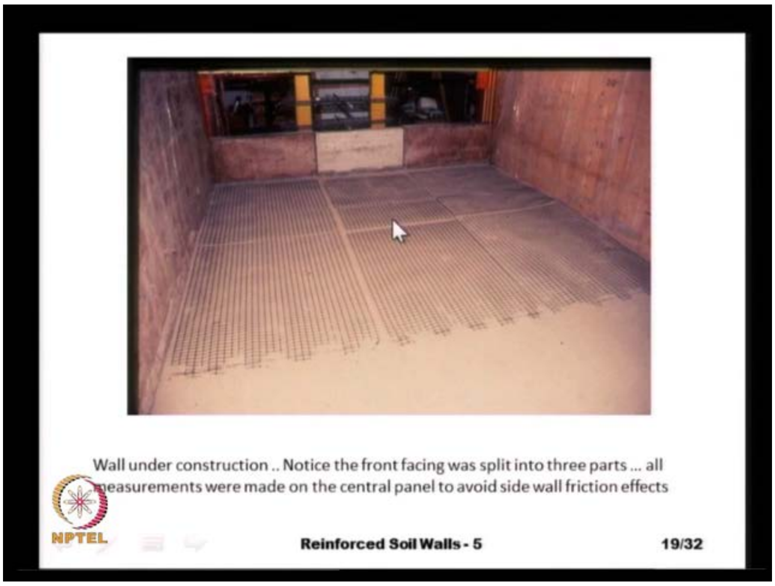

And here, in this slide, you see a large scale laboratory testing on the reinforced soil retaining walls wherein, the height of this wall is 3 meters whereas, the width is 2.4 meters. And the full scale test was done the reinforced soil retaining wall at the Royal Military College of Canada under the guidance of professor Richard Brethurst and this particular picture is from those tests.

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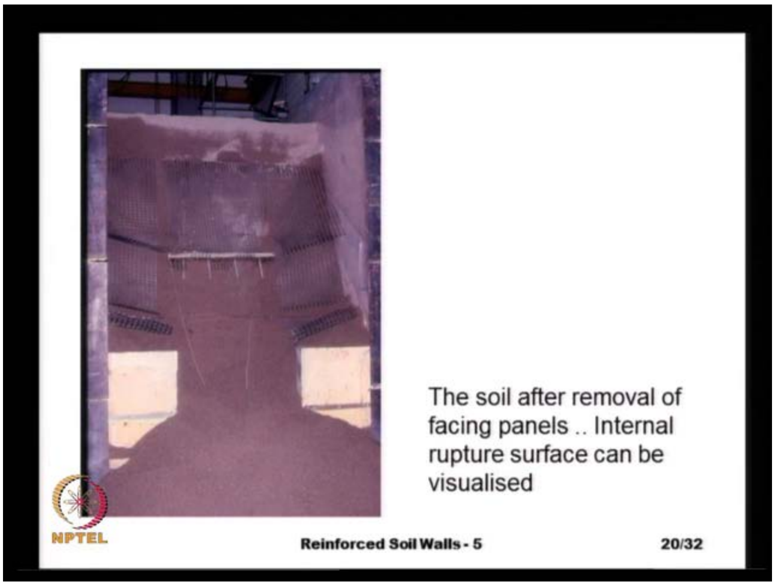

And here, we see the internal rupture surface so at the end of the test, if we remove the front panels and the soil just simply collapses the soil within the active wedge because it is already deformed sufficiently, that it just simply slides down. Whereas, the soil in the passive wedge or in the resistance zone is still standing and if we draw this line, we can complete that with the active wedge, that we get from the Rankine's theory.

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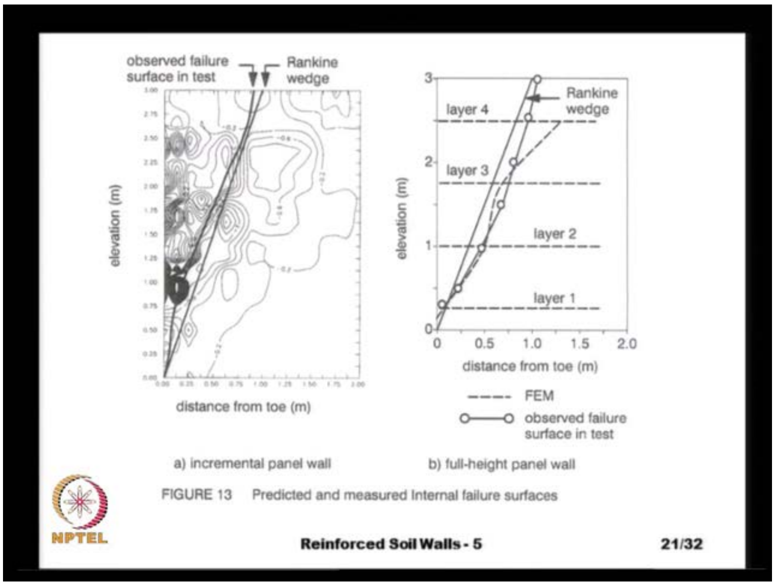

And that comparison is shown here, this straight line is the Rankine active wedge, that is drawn based on the friction angle of the soil and this is the observed failure surface from the collapsed soil after the testing was done. And on the right hand side, we also see the plot of the locations or the peak reinforcement force and there were four reinforcement layers in that particular wall, layer 1, 2, 3, 4.

And if we just simply locate the point of the peak force and join them, this is how it looks like this dotted line. And you see that, this the location of the peak reinforcement forces is also very close to the Rankine active wedge. And so that confirms that, for flexible type reinforcement layers, our Rankine active wedge can be assumed as a straight line surface.

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And for rigid materials like steel reinforcement and then the welded wire meshes and so on, lot of laboratories have done some other tests to see, what is the type of pressure surfaces that are generated. And here, you see one such facility at IIT Madras, this facility the height is 2 meters and this width is 750 milimeters and this length is 2.5 meters. And here, on the right hand side, you see one test and a progress wherein, the soil is stabilized by driving the steel nails. Nails are nothing but steel rods which are very stiff and this particular structure was subjected to failure by applying uniform surcharge through an inflatable balloon here.

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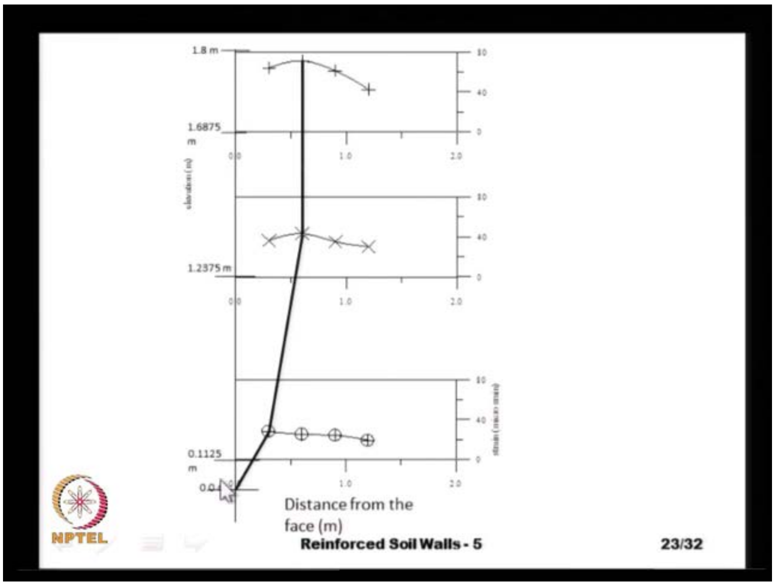

And if we plot the locations of the peak forces, it looks something like this and the surface is more of a bilinear surface, it is not a planar surface.

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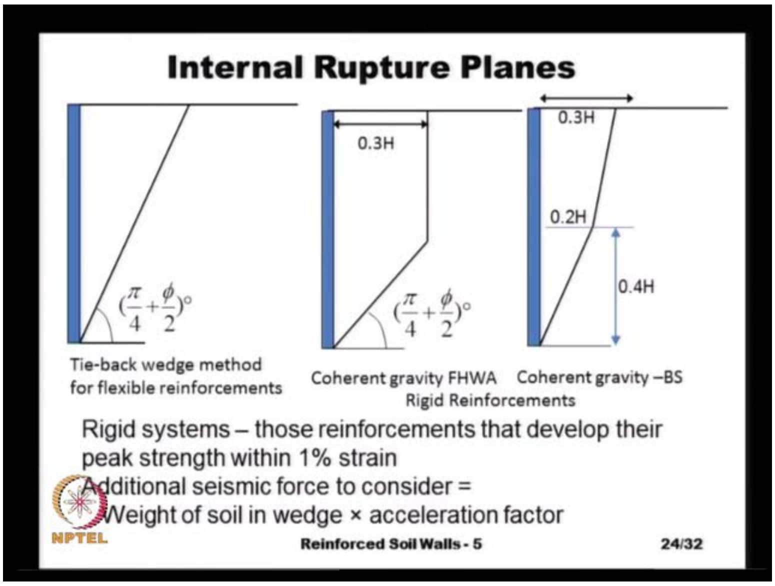

And all these results, they were compiled and put in the recommendation in different codal provisions. There are two methods of analysis, one is called as the tie back wedge method of analysis for flexible reinforcements wherein, our rupture surface is assumed to be a planar surface like this, our Rankine active surface. And so this is our active wedge and this is the passive soil or the stable soil and in the right hand side, we see bilinear wedges, this design procedure is called as the coherent gravity method of analysis.

And the federal highway administration, they recommend a wedge like this, at the top the width of this wedge is only 0.3 H and at the bottom, it is a triangular wedge like this. Whereas, the BS 8006, they recommend a slightly different surface like this, at the top of the width is 0.3 H and at a height of 0.4 H from the base, at the width of this wedge is 0.2 H. And for the purpose of deciding, which one is a rigid reinforcement and which one is a flexible reinforcement, it is given that, a rigid reinforcement is the one, in which the peak tensile force is generated at a strain less than 1 percent.

And so that draws the boundary between flexible reinforcements and there is a reinforcements or the stiff reinforcements. And all our polymeric materials, their peak force is developed at a strain of about 10 to 12 percent. So, the tie back wedge method of analysis is applicable for them wherein, our rupture surface is planar and the k is K a for the full height of the structure. Whereas, for stiff reinforcements that is, the steel strips or welded wire meshes, the analysis is coherent gravity analysis.

Wherein, our rupture surface is bilinear and the earth pressure constant k is equal to some value K naught at the top, reducing to K a at depth of 6 meters. And as per the federal highway administration, that pressure at the top is much higher about 2.5 times K a and the weight of the soil that is, within this active wedge, either this simple triangular wedge or the bilinear wedge, is used in all our seismic calculations, as we see the next slide.

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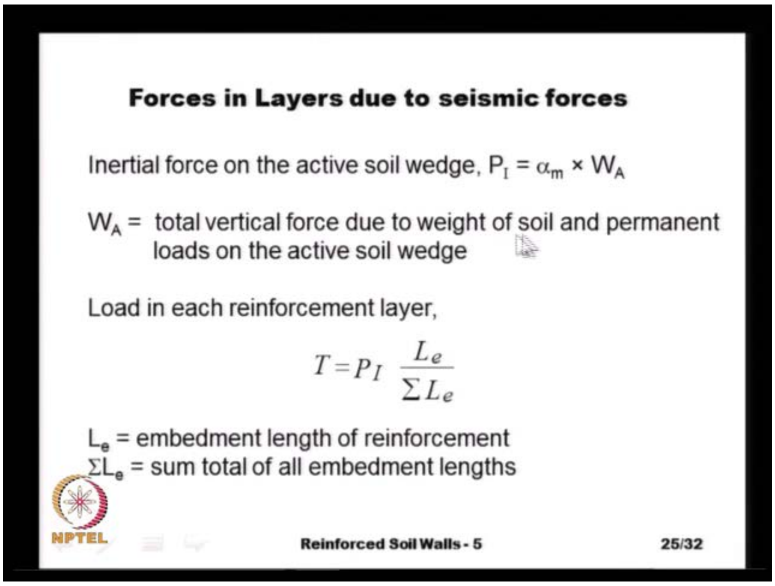

So, the forces that we consider in different layers due to the seismic inertial forces like this, the total inertial force that is acting on the soil wedge is P I that is, alpha m multiplied by W A. Where, W A is the total vertical force due to the weight of the soil within the failure wedge and then the permanent loads that are acting on the active soil wedge.

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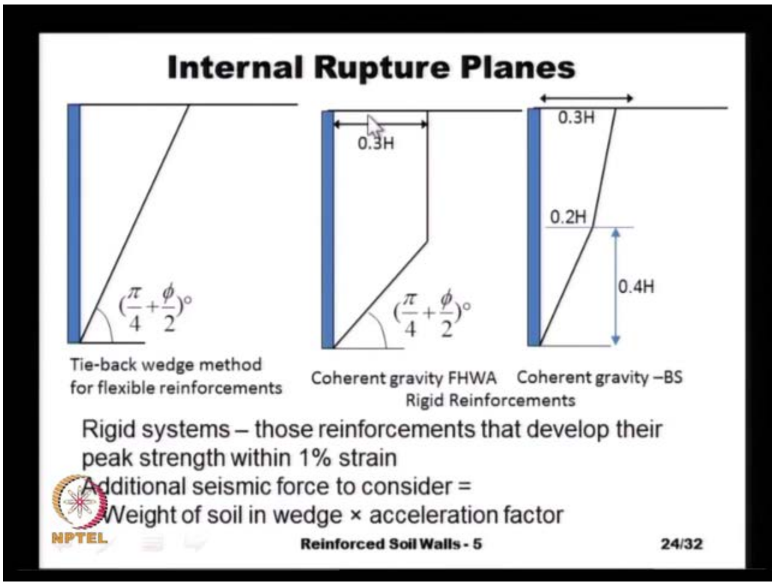

For example, for the tie back wedge method of analysis, this is the weight of the soil that we consider within this triangular wedge. And if there is any permanent surcharge that is acting on this wedge, we also take that as part of our inertial loads, and for the coherent gravity method, the weight of the soil within this bilinear wedge and then the surcharge that is acting on this some height of 0.3 H.

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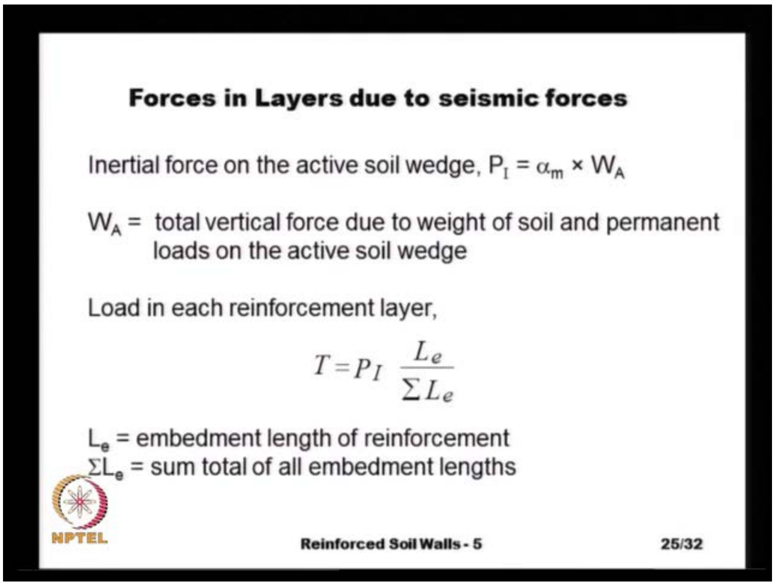

And the inertial force, that we calculate using the previous equation is distributed into each layer in proportion to the embedment length of that particular layer. And the net sum of all the embedment layers for all the reinforcement layers like shown here. The T, that is the reinforcement load in that particular reinforcement layer is, the total inertial force P I multiplied by L e divided by the sum total of all the embedment lengths, it is a very simple method of calculation. And that means that, the reinforcement layers at the bottom of the wall, they are going to carry higher inertial load, because they have much higher embedment lengths, as compared to the layers at the top of the wall.

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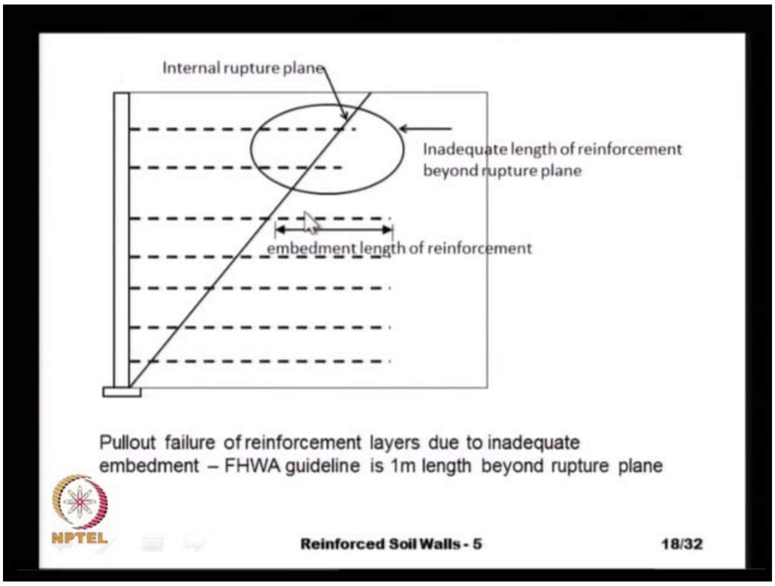

As illustrated here, we see that the layers at the bottom, they have higher length whereas at the top, they have shorter length beyond our rupture surface.

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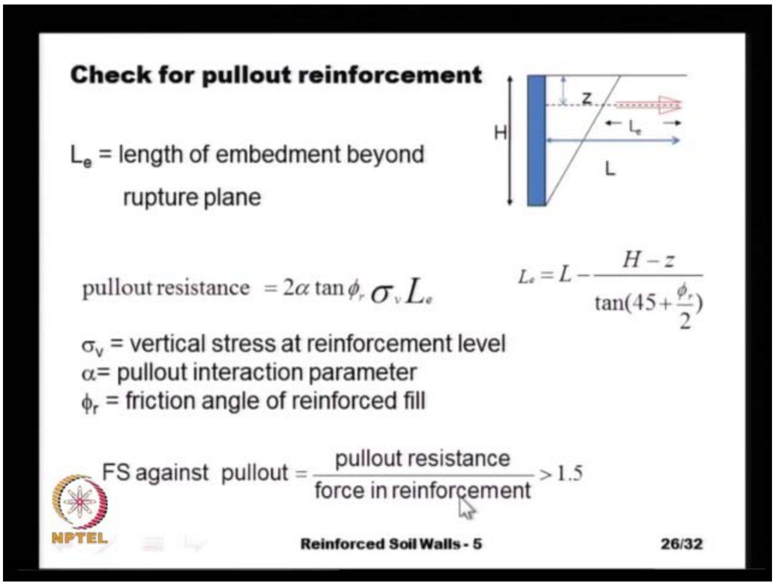

And once we are able to calculate the total force that is transferred into each reinforcement layer, we need to check for rupture by using the previous equation. And we also need to check for the pullout of the reinforcement layers for example, for the tie back wedge method of analysis, we have this triangular wedge. And our length of embedment is this and we can simply calculate the length L e as L minus, L is the total length of the reinforcement minus this distance H minus z divided by tan 45 plus phi by 2.

And the pullout resistance is given as 2 alpha times tan phi r sigma v times L e and the 2 is because we have the resistance force acting on two surfaces, the upper surface and bottom surface. And alpha is called as the pullout interaction parameter, that is determined from performing large scale pullout on the reinforcement materials and the candidate soil. And the phi r is the friction angle of the reinforced fill and the typical alpha values, they can range anywhere from 0.8 to 1 or more than 1.

And in the federal highway administration code, they give a simple formula to relate alpha to the uniformity coefficient, and that we will see later on when we go to the designs. And sigma v is the vertical pressure and when we calculate the pullout capacity, we do not use the sigma v b that is, may half pressure. But, we just simply use the sigma v that is, the static pressure multiplied by this L e and the minimum factor of safety against the pullout failure is atleast 1.5, that is the ratio between the pullout resistance calculated by this formula divided by the force transferred into each reinforcement layer.

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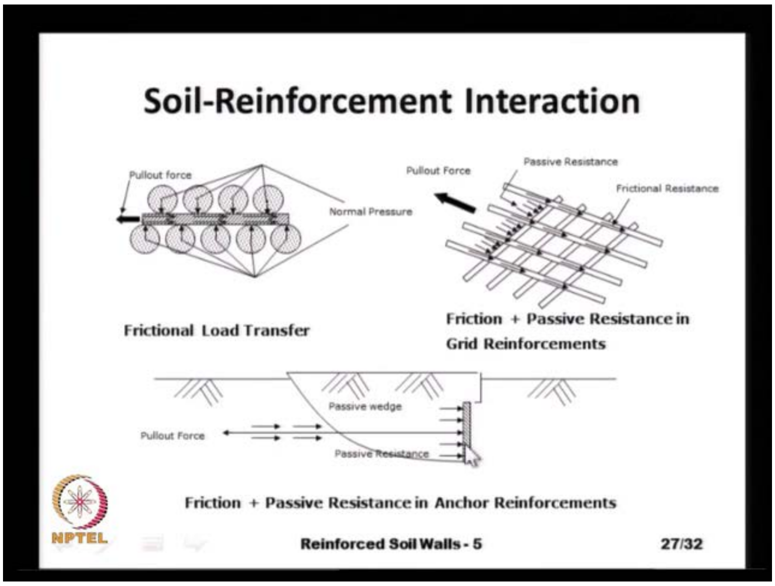

And the pullout capacity itself is mobilized, because of several reasons in different type of reinforcement materials. Say, geotextiles or steel strips are the reinforcement pullout capacity is only, because of the frictional load transfer that is happening along the length of the reinforcement, both at the upper surface and bottom surface. Whereas, for geo grids or the welded wire meshes, we have number of these cross members that can develop lot of passive resistance as indicated here.

And so here the pullout capacities, because of the friction that is developed along the surface plus the passive resistance of these cross members. And when we have the reinforced anchor elements and the pullout capacity is, partly because of the friction that is developed along the length of the reinforcement, and partly because of the passive resistance that is developed because of this vertical anchor.

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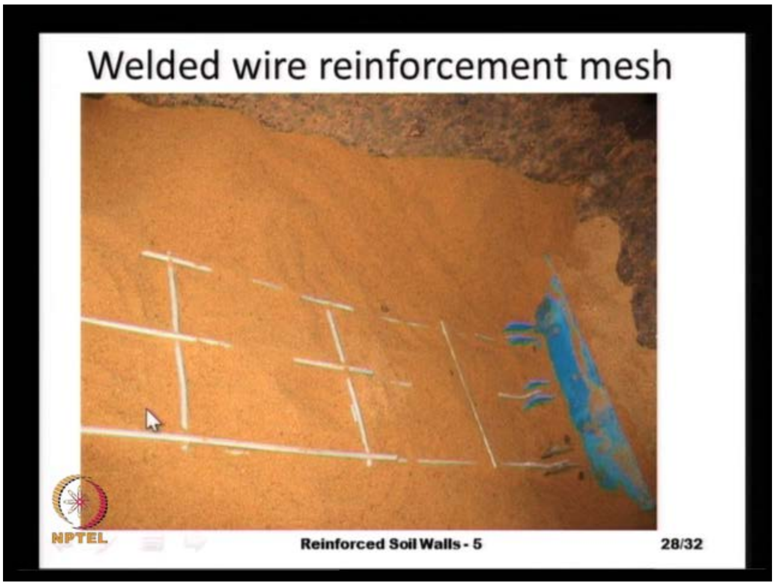

And the welded wire mesh, typical welded wire mesh is as shown here, we have length and then we have this cross members, against which some passive force can be developed.

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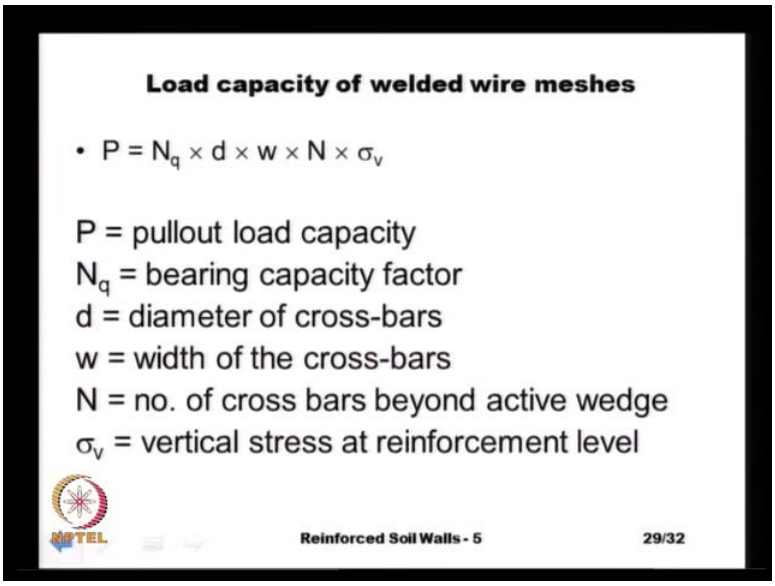

And the pullout load capacity of the welded wire meshes is estimated using this type of formulas, N q times d times w times N times sigma v where, N q is our bearing capacity factor, very similar to our bearing capacity equations. And the d is the diameter of the cross bars and w is the width of the cross bars, N is the number of cross bars with beyond the active wedge that is, within the stable soil zone and sigma v is the vertical stress that is acting at the reinforcement level.

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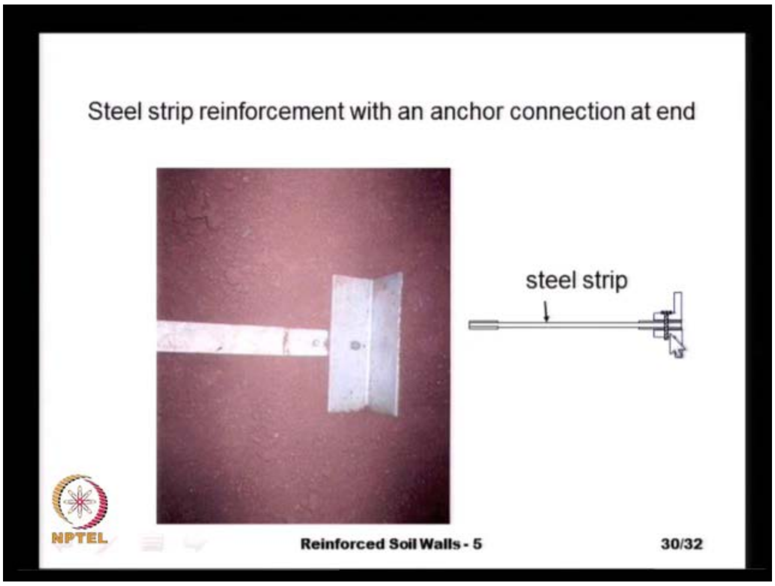

And a typical picture of a steel strip reinforcement with anchor is shown here, the reason why we use an anchor for steel strips is, as you know the steel has very high yield strength and compared to the pullout capacity is very low. To increase the pullout capacity, we can add an anchor element here, in this particular case, the anchor element is made up of a L angles, L sections and it is directly connected to the steel strip like this.

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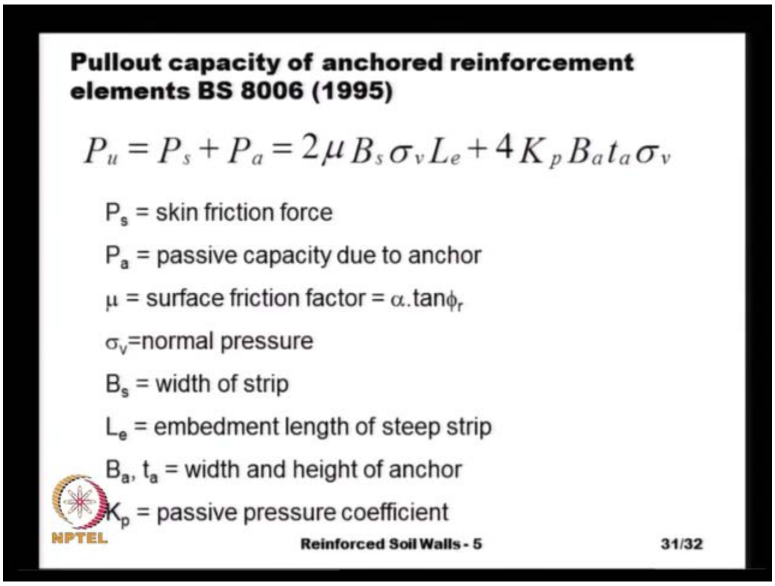

And the BS 8006, they give a very simple formula to estimate the pullout capacity of the anchored reinforcement elements, as the sum of the skin friction capacity plus the P a that is, passive capacity due to the anchor. And the skin friction capacity is written as 2 times mu b s sigma v times L e where, mu is the surface friction factor which is similar to our alpha times tan phi, that we have seen in the earlier equation.

And the B s is the width of the steel strip and sigma v is the vertical pressure, L is the embedment length and the capacity, because of the anchor is written as 4 times K p times B a t a times sigma v. Where, K p is the passive earth pressure coefficient, B a and t a are the width and height of the anchors, sigma v is the vertical pressure. And once we are able to satisfy the adequate factors of safety against rupture and the reinforcement, we also need to check for the connection failures, that we will see in some other lecture.

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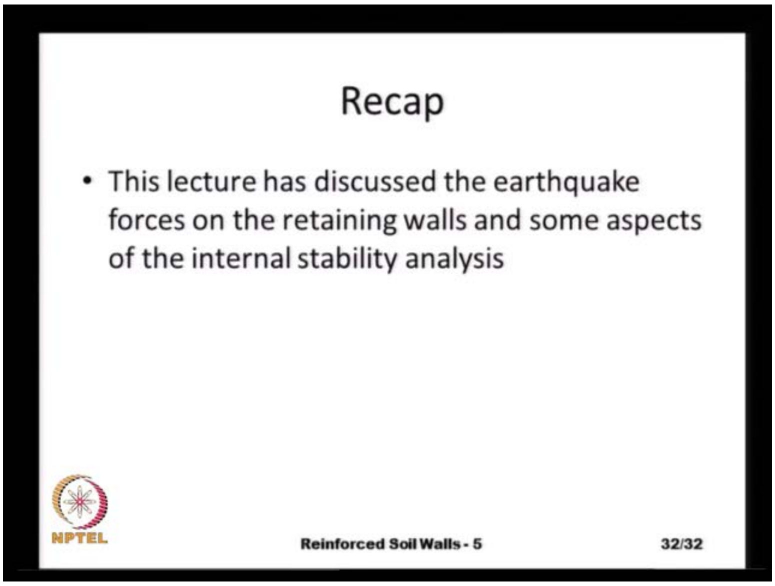

And just briefly recap, we have looked at how to calculate the additional forces that are induced in our reinforced soil structure, because of the earthquake. And then some aspects of the internal stability analysis are discussed.

Thank you very much.