Geosynthetics and Reinforced Soil Structures
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Lecture - 22
Geosynthetic Reinforced Soil Embankments - II

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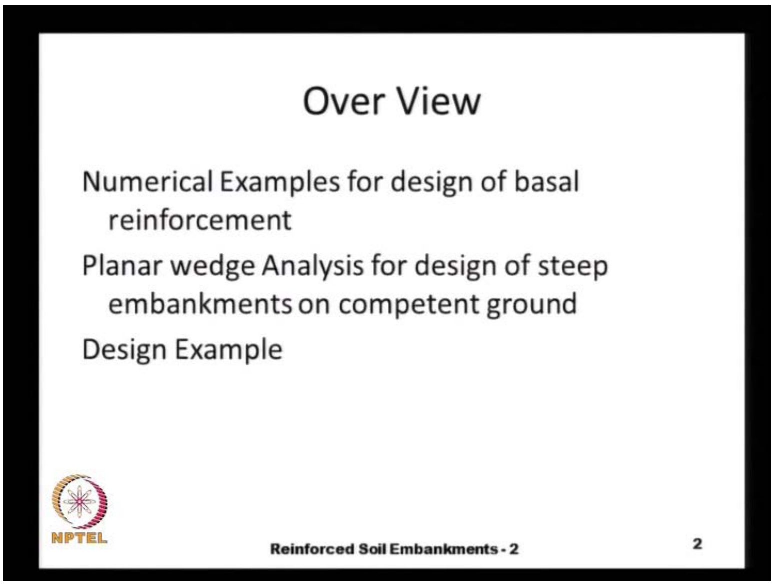

Very good morning students. In this lecture, we will go through some numerical examples for the design of basal reinforcement that we had studied in the previous lecture. And then the planar wedge analysis for design of very steep embankments on competent soils and some design examples.

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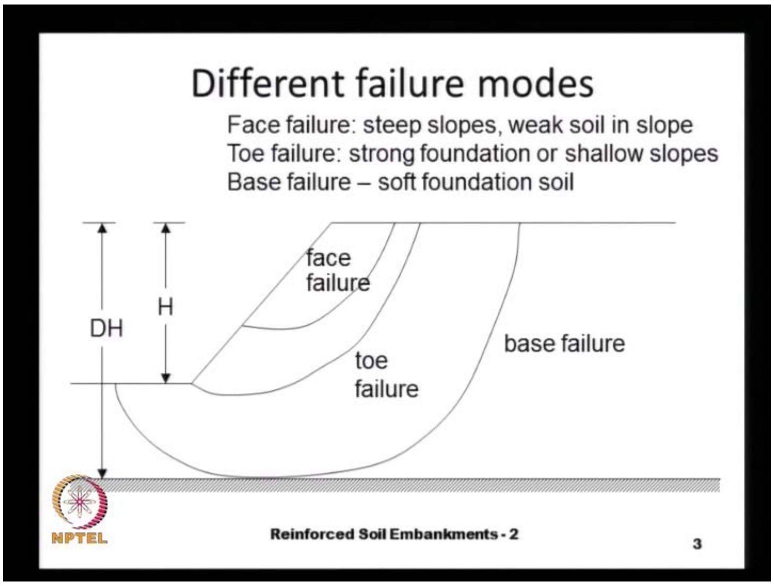

As you may recall from your other geotechnical engineering courses these are the different types of failures that we can have with these slopes. Let us say that you have a slope of height H and there are three possible modes of failure; one is the face failure the face failure happens in the case of steep slopes or if the embankment is formed with a relatively weak soil. And then the toe failure is is the failure that happens when we have a strong foundation soil or very, very shallow slope then base failure or deep seated failure happens when we have a an extremely soft foundation soil, and our embankment soil itself is relatively strong.

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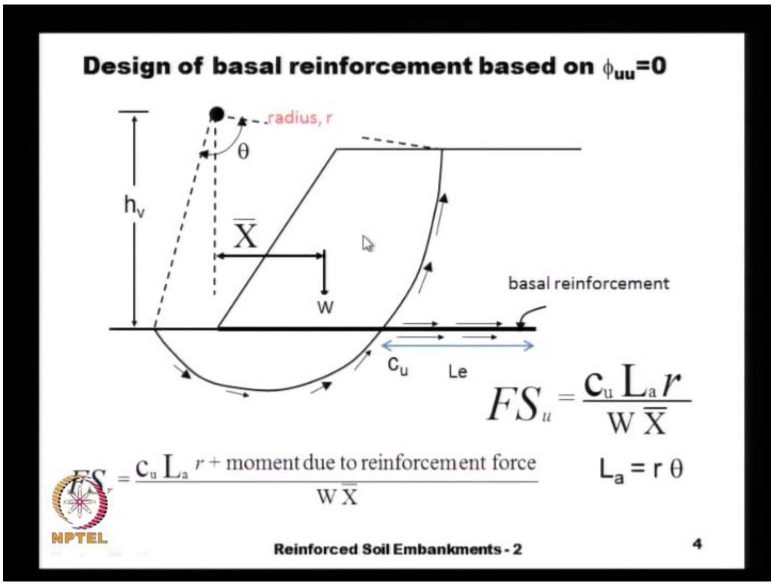

And just to recall in the previous lecture we have seen the the design of soil slopes with basal reinforcement layer, and the basal reinforcement it is provided for several reasons especially in the case of extremely soft case we require a working platform. So, that we can even bring in our construction machinery and start mobilizing our equipment and then of course, on top of that we may require some extra reinforcement to improve the factor of safety and for a very simple case where our phi equal to 0.

We can get the factor of safety for a given circle a by just taking moments about this centre of rotation, and w is the weight of the soil within this wedge and X bar is the is the lever arm W times X bar is the destabilizing moment. And then the stabilizing moment is because of the the cohesive force that is acting along this slip circle length that is a C u times L a where L a is the length of the slip circle multiplied by this r radius r and that is the factor of safety of the unreinforced slope. And if you have a basal reinforcement we need to add extra stabilizing moment because of the basal reinforcement and. So, we have seen two cases of basal reinforcement one is a very stiff reinforcement that remains horizontal and another is flexible type reinforcement that bends that has a kink at this slip surface in which case the axial force in the reinforcement as tangential to the to the slip surface.

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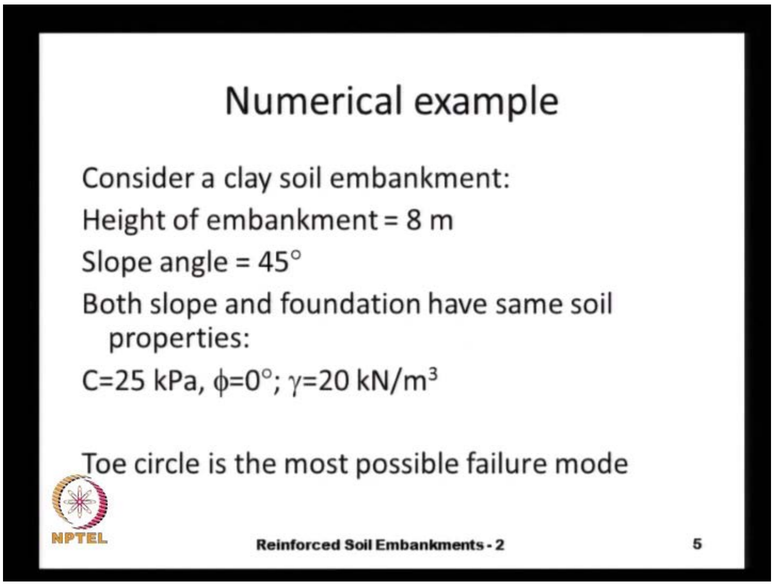

And now a let us look at a numerical example let us consider a a clay soil embankment height of a 8 meters, and the slope angle is 45 degrees. And let us say that both the slope and the foundation soil have the same soil properties of C u of 25 k P a, and friction angle is 0, and unit weight is 20 kilo newtons per cubic meter and this situation a toe circle is the most possible failure mode. And as we discussed in the previous lecture we need to consider several of these slip circles to to get the circle with the least factor of safety, and whatever equation that you have seen is only for one single slip circle and we need to to try out several of these to get our minimum factor of safety.

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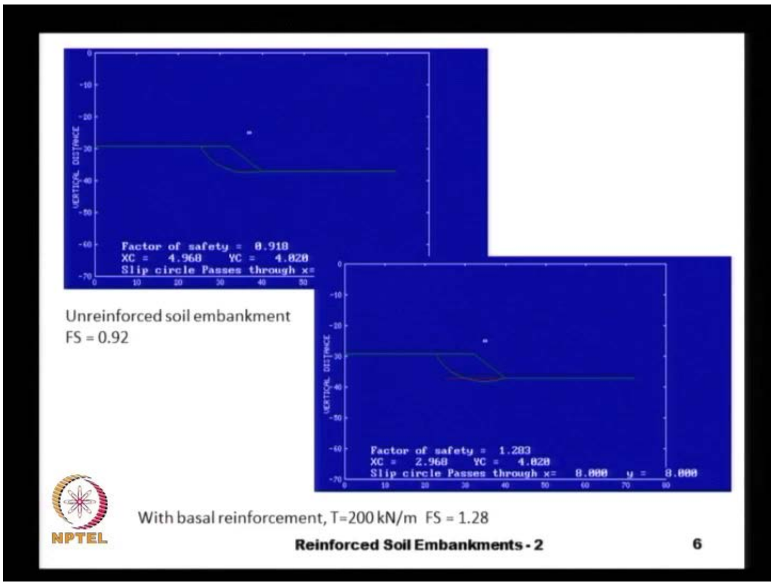

And it is best done using some computer program, and in this case this is the result from one computer program that you are going to get as part of this course the height of the slope is 8 meters, and this slope angle is 45 degrees. And the the factor of safety of this unreinforced slope is 0.92 and we need to increase this factor of safety to a higher value and that is a trial and error process, and here we have the result from another slip circle analysis with basal reinforcement having a tensile strength of 200 kilo newtons per meter. And for this case the factor of safety is is come up to 1.3 and we may need to choose basal reinforcement provides higher force in case, if you want factor of safety higher value and the length of this reinforcement that we need to provide should be adequate. So, that the the it has sufficient pullout resistance and the pullout resistance is calculated based on the embedded length of reinforcement beyond the the most critical slip circle, and these are all based on using some programs. Because doing the hand calculations is quite tedious, because typically we look at least 2 to 300 slip circles for coming out with these minimum factor of safety, and in this case as I mentioned earlier the, if you are not happy with this factor of safety we need to increase the increase the a tensile strength of the reinforcement. And typically we do not provide one single layer as illustrated here we may provide some two or three layers, so that we can achieve our target factor of safety.

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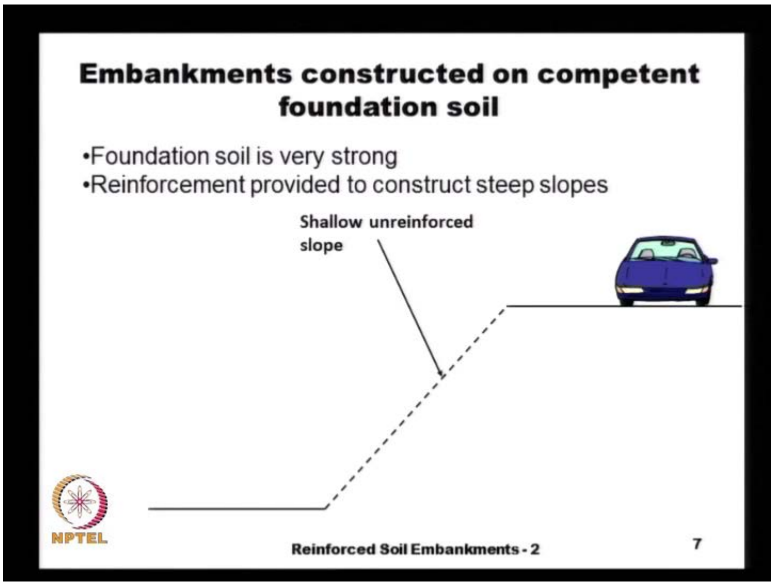

And continuing our discussion many many times, when we have road embankments and other things in the city areas we may be faced with a requirement that we need to construct a very steep slopes, and one example as we had seen earlier is is like this.

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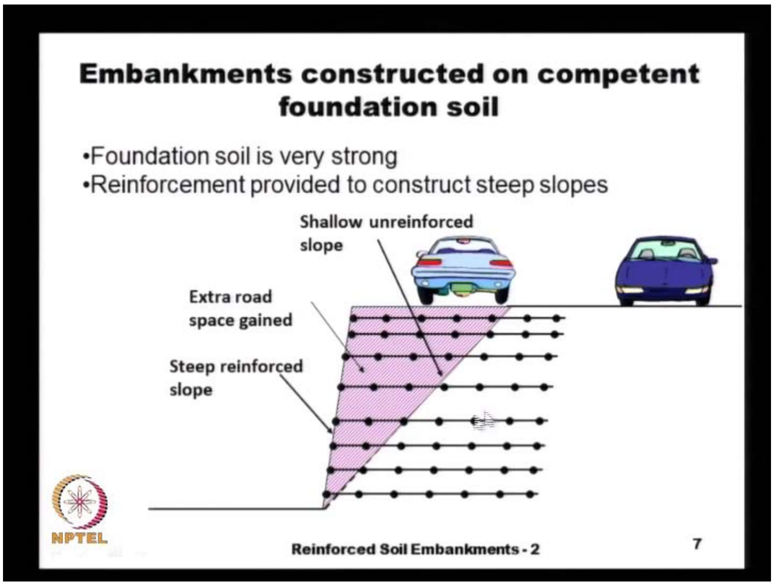

Let say that we want to widen this road and the only way to widen this is either to make this slope as a steep slope, and so that we get extra space or or procure more land and and widen the construction area and in the city urban areas getting extra land for our construction is very difficult. So, in that case we going for going for steepest slope by providing horizontal layers of reinforcement.

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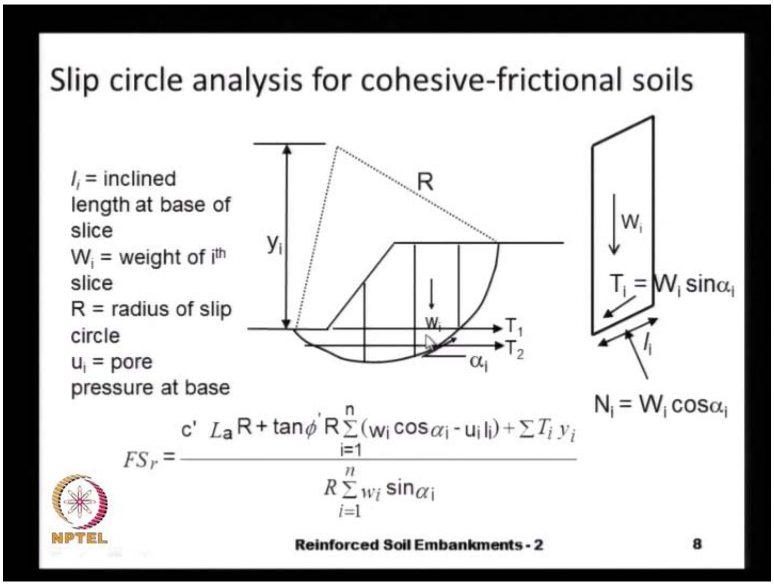

And how do we design this type slopes there are different design approaches one is the slip circle analysis, we can just do a the felonious or bishop’s slip circle analysis and find out the factor of safety. And here this equation is from the felonious analysis or the ordinary method of slices once again we take moments about about the centre of rotation and for the convenience, we divide a this entire length of this slip circle into into certain number of slices. And typically at least minimum 6 to about 10, and we consider the equilibrium of each slice W i is the weight of the soil that is acting down, and that causes some shear force T i, that is trying to destabilize this wedge. And that force is T i time W i times sin alpha i that is just by resolving this force into into parallel to the to this to this base and perpendicular to this base.

And the perpendicular component N i the normal component is W i times cosine alpha i, and let us say that i length of this base is L i that is the incline length. And we can calculate the factor of safety of the reinforced slope as the resisting moment divided by the upsetting moment. And the resisting moment is because of two components, one is because of the cohesion. And the other is because of the friction angle and the third one is because of the the reinforcement that we have, and say if you are working with effective stresses C prime L a, that is the length of this slip circle r multiplied by r that is the the the lever arm for this cohesive force. And tan phi times N i prime and N i prime is a nothing but W i cosine alpha minus u that is the pore pressure multiplied by base angle L i. And this pore pressure is we can estimate it, if we have the a freeatic surface or in terms of the bishop's pore pressure parameter, we can as estimate u as some R u times W. And this entire quantity that we have w times cosine alpha minus u i L i is similar to our effective stress and this whole thing multiplied by r that is the lever arm. And see we have certain number of reinforcement layers, we can take the summation of t times y and that the the resisting moment divided by the upsetting moment.

Because of this component of the force T i is the W i sin alpha i will give us the factor of safety of the reinforced slope, and this method is recommended in the in the federal highway administration code, and even in the b s code they recommend, but its only for preliminary analysis.

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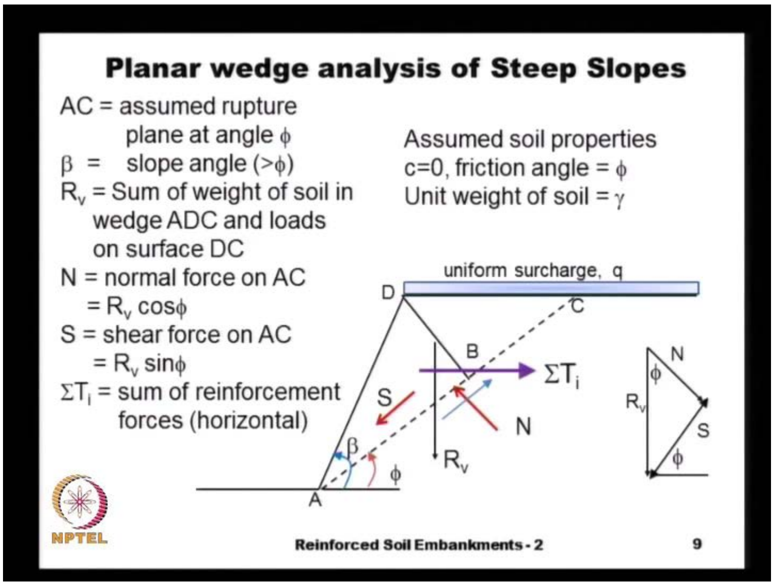

And we have more number of other analysis that we can consider one simple method is the planar wedge analysis of the steep slopes, we will understand the principle of the slope stability analysis with this. And we will go on for looking at more complicated surfaces. Let us say that we have a slope within angle of beta, and then we want to analysis and we have provided certain number of reinforcement layers, and and the simplest one is, if we assume that the failure angle is at an angle of phi that is the angle of repose; that is the simplest one to consider. And let us say that there is a uniform surcharge of q and the failure is happening along the surface AC, and let us assume that our cohesive strength is C is 0. And the friction angle is phi and the unit weight of the soil is gamma and let us say that this is the AC is the assumed rapture plane at the angle of phi and beta is the slope of an angle. And beta is much greater than phi and RV is the sum of a the weight of soil within this wedge ADC and the loads acting on the surface DC. And n is the normal force on the AC that is just simply R v times cosine phi that is if you can see this force triangle R v is the vertical force n is the n is the normal component. And let us say that the sum of all the the reinforcement forces is sigma T i that is the sum of all the the reinforcement force that we have.

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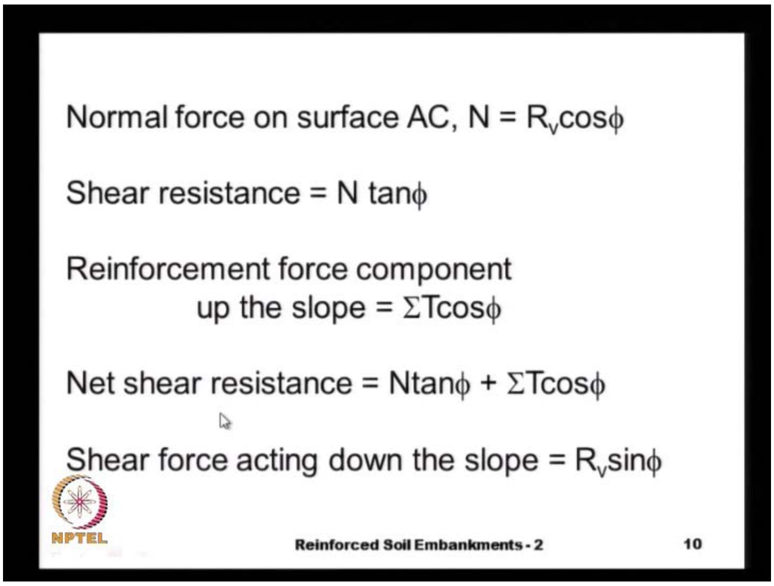

And we can do the equilibrium analysis of all the forces that are acting on the on the failure surface AC, as we have seen earlier the normal force is R v times cosine phi. And the shear resistance is n times tan phi, because the friction here is because between two soil surfaces, because here this this slope AC is the rapture surface on both sides we have this the soil. And we can assume that the friction angle mobilized along that surface is only the phi and the reinforcement force component upwards of the slope is just simply sigma of t times cosine phi that is the the component of the reinforcement force, and then the net shear resistance is n times tan phi plus sigma t cosine phi. And the shear force acting down the slope is R v times sin phi, that is that is the shear force that is acting down here.

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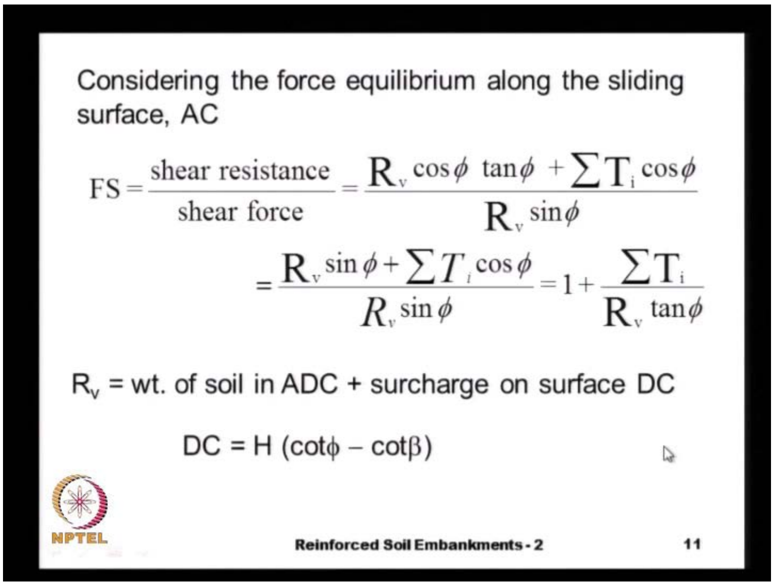

And if we consider the force equilibrium of all the forces acting along the sliding surface AC out factor of safety is the shear resistance divided by shear force, that is R v times cosine phi times tan phi plus sigma of T i cosine phi this whole thing divided by R v sin phi and. So, cosine phi times tan phi is just simply sin phi. So, it is equal to R v sin phi plus sigma t i cosine phi by R v sin phi that is 1 plus sigma of T i R v tan phi by bringing this cosine phi to the denominator. And R v is as we discussed earlier the weight of the soil within the triangle a d c plus the surcharge, that is acting on the surface D C, and this length D C is H times cotangent phi minus cotangent beta.

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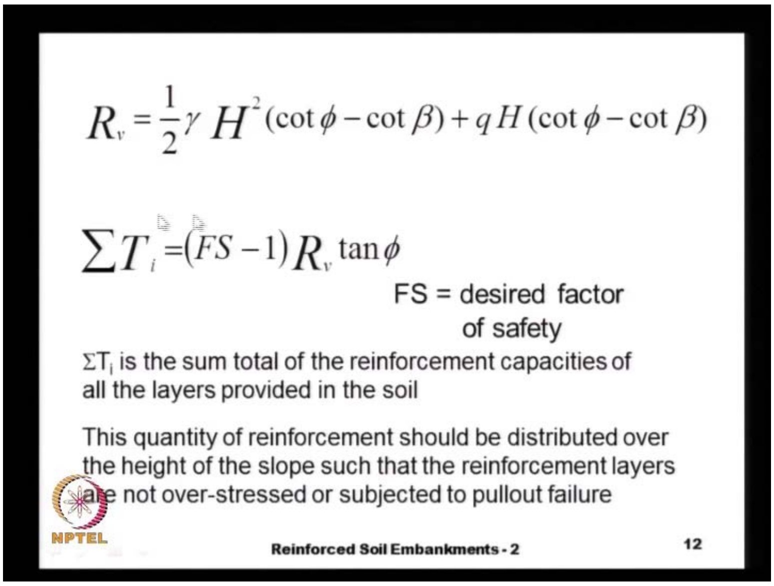

And our R v now is just simply one half gamma square times cotangent phi minus cotangent beta plus the surcharge force acting on the surface D C; that is q times H cotangent phi minus cotangent beta. And so our factor of safety equation F S is one plus sigma T i by R v tan phi and by rearranging that equation. We can get our sigma T i in terms of the factor of safety and the other fact is as F S minus 1 R v times tan phi where in our F S is the desired factor of safety.

And this one is the factor of safety for soil to soil slope failure, and because we have assumed that the slope failure is along this this angle of repose that factor of safety is one, and this F S is the the the desired factor of safety. Because of the provision of reinforcement layers our factor of safety is going to greater than 1, and so this sigma T i is the sum total of the reinforcement capacities of all the layers provided in the soil to increase the factor of safety of the slope from one to F S. And so this quantity of reinforcement should be distributed uniformly over the height of the soil of the the soil such that the reinforcement layers are not over-stressed or subjected to pullout failure over-stressed means the force; that is transferred into each reinforcement layer should be lower than the tensile strength with some adequate margin of safety.

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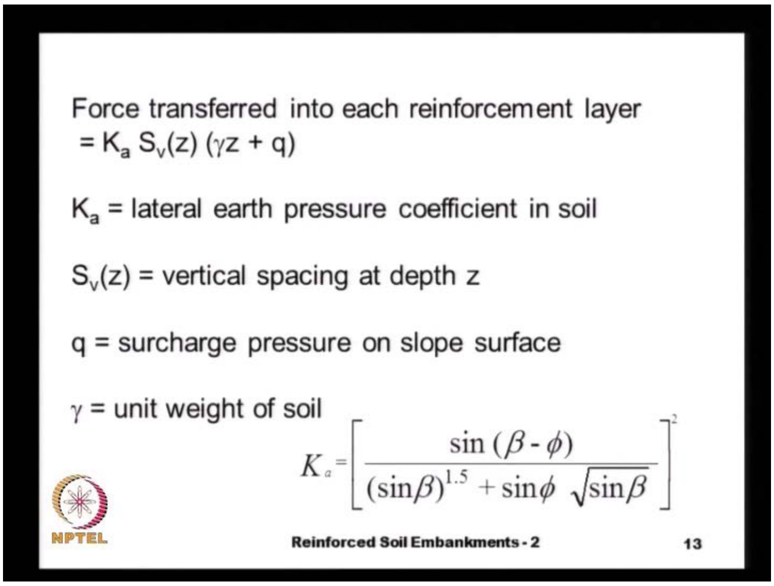

And as we have seen earlier in the case of reinforced soil retaining walls the force transferred into each reinforcement layer at a depth of Z is K a times S v of Z multiplied by by the vertical pressure gamma Z plus q in the case of retaining walls. We have considered a mayer half pressure distribution, but in the case of slopes we just considered only the vertical stress without any effect of overturning here our K a is the lateral earth pressure coefficient in the slope.

But this is not equal to the rankine's at the coulombs earth pressure coefficient is much lower, because this K a is operating in a in a shallow slope, and S v of Z is the vertical spacing at depth of Z. And the q is the surcharge pressure acting on the slope surface and gamma is the unit weight, and this k a is is the sin beta minus phi divided by sin beta to the power 1.5 plus sin phi square root of sin beta to the whole of square. And this quantity if we substitute beta of 90 degrees will be equal to our K a for rankine theory that is 1 minus tan phi by 1 plus sin phi. And for the slopes this is much lower as we will see later on.

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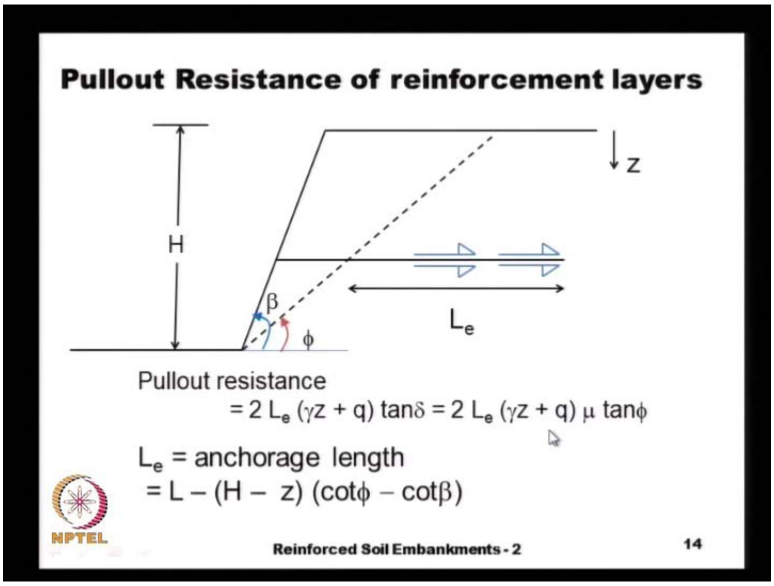

And the pullout resistance is estimated based on the length of the reinforcement beyond the rapture plane like this. So, in our case our rapture surface is at an angle of phi like this and let us say our reinforcement length total length is l out of that l e is the anchorage length. And this anchorage length l e is l minus minus, this length that length is H minus Z times where Z is the the height from the top cotangent phi minus cotangent beta. So, this L e if we have it, because our pullout resistance is mobilized on two surface we have a two here multiplied by L e; that is the anchorage length times. The vertical stress gamma Z plus q times tan delta that is the delta is the the interface friction angle or we can also write it as sum nu times tan phi, because where mu is called as the interaction factor and the pullout modular failure.

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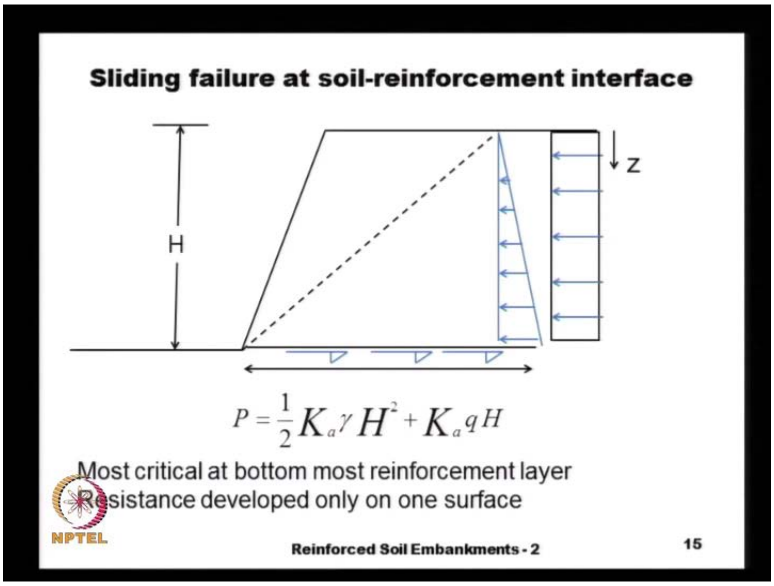

And phi is the friction angle, apart from second for the for the reinforcement rapture and the pullout we also need to look at some other modes of failure; that is let us say that this entire length of soil slope behaves like rigid body. And it is subjected to lateral earth pressure from behind, that is one is because of the self weight of the soil that has triangular pressure distribution, and the other because of uniform surcharge that has a rectangular pressure distribution and this rigid block of reinforced soil may slide away by sliding above the the bottom most reinforcement layer.

And the in this case our shear resistance or the base resistance is developed only on one surface like this and our total lateral force is one half K a gamma H square plus K a times q times H where our K a is the reduced earth pressure coefficient. As we have seen earlier and the most is actually this failure may happen at any depth, because wherever we have a reinforcement ah that may be the weakest plane, because the the friction angle that is mobilized between the soil and the reinforcement is usually slightly less than phi.

And we need to theoretically do the check at each and every reinforcement layer, but then the most critical one is at the bottom because the lateral force is the maximum at the at the base. And if our reinforcement wedge at different at different depths then we may have to check ah at all the depths, but in usually we provide equal length of reinforcement in the slopes because the from the construction point of view that is more easy.

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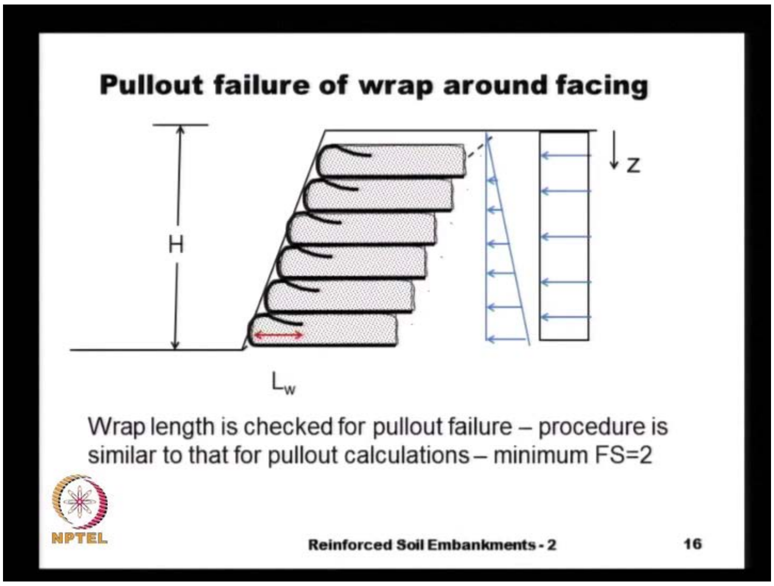

And apart from this the other earlier failures that we have considered whenever face is made up of wrap around facing especially with geogrid or geotextiles this length of wrap around we should check to make sure that there is good anchorage. If that wrap around length is not sufficient it will just get a pushed out and then we may have a face failure. And the minimum factor of safety that is required for this for this mode of failure is two and for this, what we do is? We assume that this particular wrap around is resisting the force that is coming from this height of the soil, and then equate the at the pullout resistance of this reinforcement length to this lateral force to make sure, that we have adequate embedment length and if it is not, it is not adequate. Then we will have to increase the embedment length, and typically this embedment length is at least 1 1 meter.

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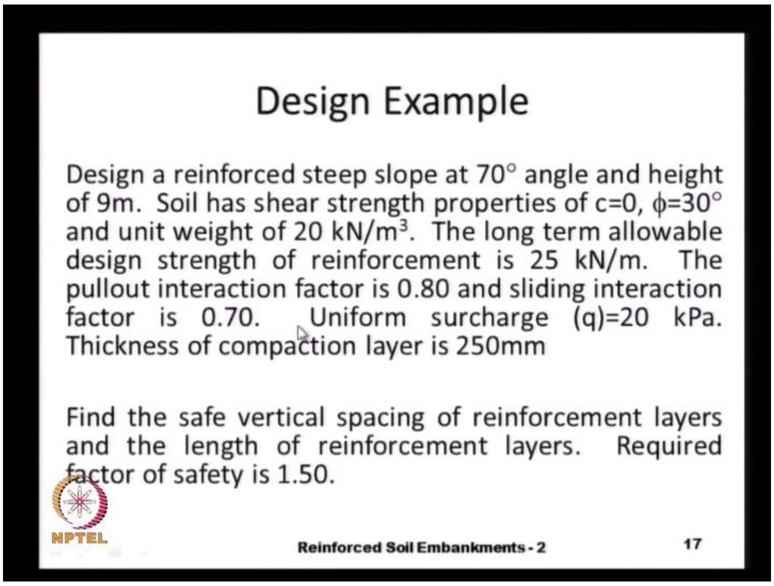

So, now let us consider a a simple example let us design a reinforced slope which has 70 degrees angle, and the height of the slope is 9 meters. And the shear strength properties of the soil or C equal to 0, and friction angle phi is 30 degrees and the unit weight of the soil is 20 kilo newtons per cubic meter. Then the long term allowable design strength of the reinforcement is a 25 kilo newtons per meter and the pullout interaction factor is 0.8 and the sliding interaction factor is 0.7, and the uniform surcharge is 20 k P a and then the thickness of the compaction layer is 250 millimeters. And as we have seen with the design of reinforced soil retaining walls the vertical spacing between the reinforcement layers should be multiples of the thickness of this compaction layer, and we are required to find the safe vertical spacing of the reinforcement layers, and the length of reinforcement layers and our required factor of safety is 1.5.

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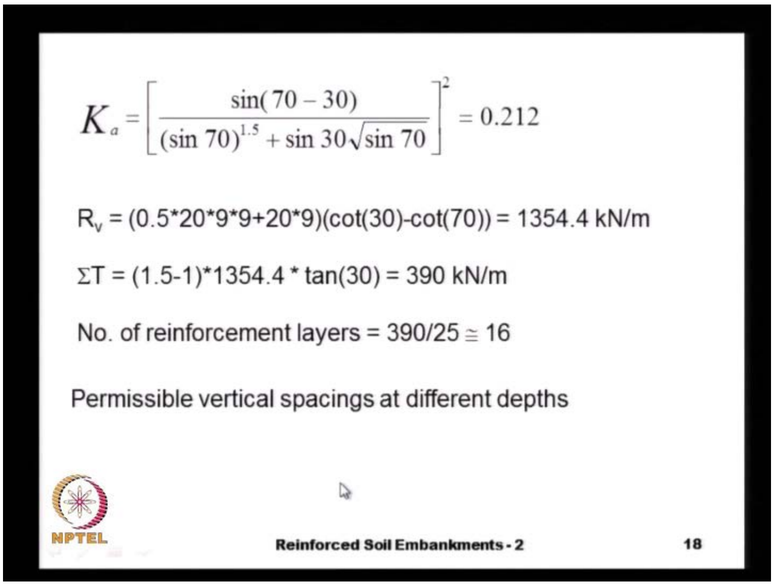

So, now let us go back to our equations and our K a for this case is if we substitute beta of 70 degrees and friction angle of 30 degrees into that equation, we get 0.212. And for a vertical soil retaining wall having a friction angle of phi our K a is one third that is 0.33 whereas, here this value much lower 0.212, and our R v that is the the vertical force acting in that triangular portion that is the sum of the soil weight. And then the surcharge load is one half gamma H square plus q times height H and cotangent 30 that is phi minus cotangent beta that is cotangent 70, that is 1354.4 kilo newtons per meter and as you may recall all our designs are done per unit length in the perpendicular direction to the plane of analysis.

And if you substitute this our sigma of t that is a sum total of our reinforcement forces comes out as 390 kilo newtons per meter, and our number of reinforcement layers is 390 by 25 that is approximately 16. And our 25 is the long term allowable design strength and this long term allowable design strength should include the construction induced damage biological damage then chemical degradation. And then the cre-prediction factor as we have discussed in earlier lectures, and so this the total reinforcement force required is 390, and the number of reinforcement layers is 25 sorry 16, and now we need to find the permissible vertical spacings at different depths.

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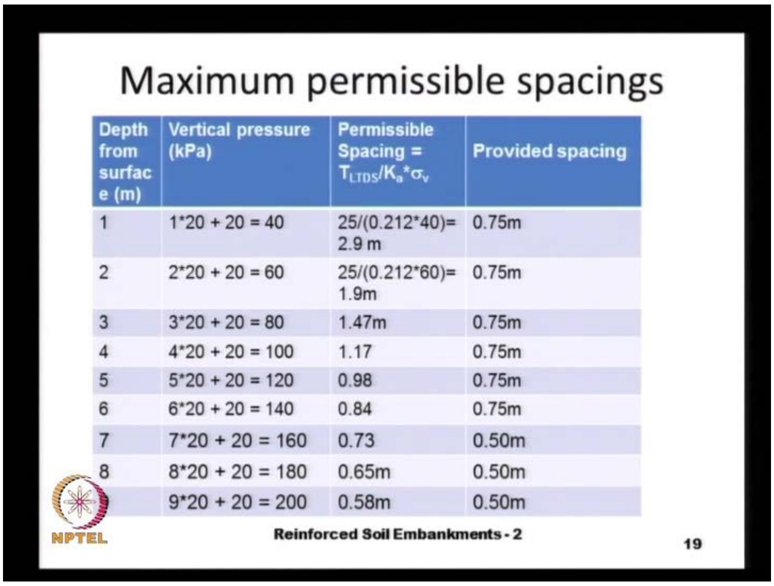

Maximum permissible spacings

Depth from surface (m)	Vertical pressure (kPa)	Permissible Spacing = $T_{LTDS}/K_a*\sigma_v$	Provided spacing
1	1*20 + 20 = 40	25/(0.212*40)= 2.9 m	0.75m
2	2*20 + 20 = 60	25/(0.212*60)= 1.9m	0.75m
3	3*20 + 20 = 80	1.47m	0.75m
4	4*20 + 20 = 100	1.17	0.75m
5	5*20 + 20 = 120	0.98	0.75m
6	6*20 + 20 = 140	0.84	0.75m
7	7*20 + 20 = 160	0.73	0.50m
8	8*20 + 20 = 180	0.65m	0.50m
[illegible]	9*20 + 20 = 200	0.58m	0.50m

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Reinforced Soil Embankments - 2

19

And for this our permissible spacing is the T, that is the a the long term allowable design strength divided by the the force transferred over height of S v, that is the the spacing and that spacing. If we equate T 2 that permissible spacing in terms of s v our spacing comes out as T by T a times sigma v, and this we need to do at at all the depths at some convenient depths. So, at a depth of 1 meter the vertical pressure is 40. So, it is 25 by this quantity 0.212 times 40 and this comes out as a 2.9 meters, but then our if we provide reinforcement at a very large spacing, we may have failure between two reinforcement layers and because of that all the design codes they recommend the maximum spacing.

That you can provide usually it is not more than one meter because we have very large spacings each the soil height between the reinforcement layers may have failure because as you recall the action the reinforcement is only in its own plane, and the affect gets transferred to some height. So, here the last column is the provided spacing that is 0.75 meters and this 0.75 meters is in the multiples of the in the thickness of the compaction layer is 250 millimeters. So, if we do this calculations at a depth of two meters it is 1.9 at a depth of three meters 1.47 and it go it is reducing at the bottom most 0.9 meters it is 0.58 meters.

So, theoretically we cannot provide the 0.58 because our compaction layers or in a thickness of 250 mm. So, we need to provide at minimum 50 milli 50 centimeter spacing and as we go up even at a depth of 7 meters the spacing required is 500 m m and at a depth of 6 meters our permissible spacing increases to 840 m m. So, we can provide at 750 m m. And now we need to a distribute these 16 layers or the height of soil by taking this as the guideline the maximum spacing that we provide at any depth should not exceed these values.

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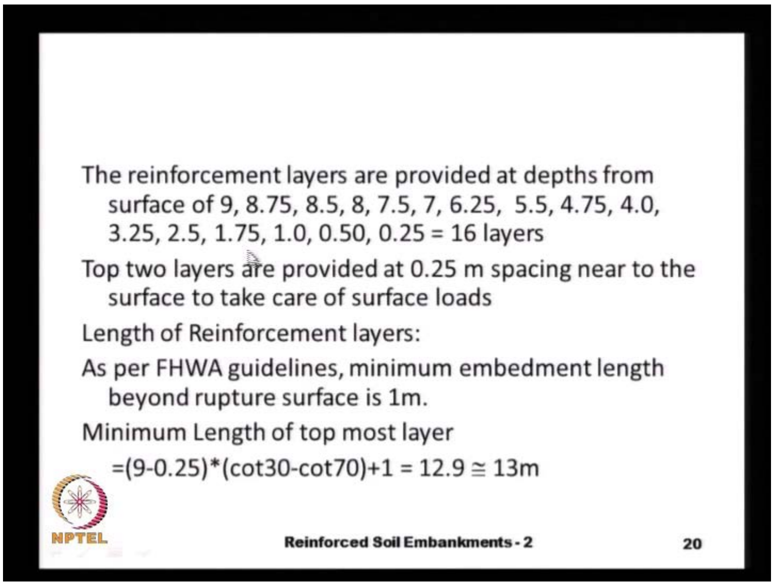

So, by trial and error we can provide the 16 layers this particular case we can provide one reinforcement layer at the at the base that is at 9 meters depth, and another at two 50 m m 8.75, 8.5 8, and so on. And towards the top we need one extra layer of reinforcement, because that is to take care of any surface compaction pressure and then the surface traffic that is moving on and so this the we have provided as per the codal provisions the top most layer should be at a depth of one compaction layer.

So, that is a 250 m m in this case and is actually these provided spacings or slightly less than 500 m m, that is 250 m m to accommodate 16 layers. And now we need to once we choose the vertical layout, now we need to choose the the length of reinforcement layers. And the length of reinforcement layers is very simple to calculate if you understand that we need a certain minimum anchorage length, and the federal highway administration guidelines say that minimum embedment length is 1 meter. And that means, that the in terms of the criticality the top most reinforcement layer is the most critical. Because that will have the least embedment length assuming that all the reinforcement layers are of the same length. So, this calculating the length of the top most layer that is 1 meter is the is the the length of reinforcement beyond the rapture surface. And the distance from the slope up to the to the rapture surface is 9 minus 0.25; that is H minus Z time cotangent phi minus cotangent beta plus 1 that is 12.9, that is you can round it off 13 meters.

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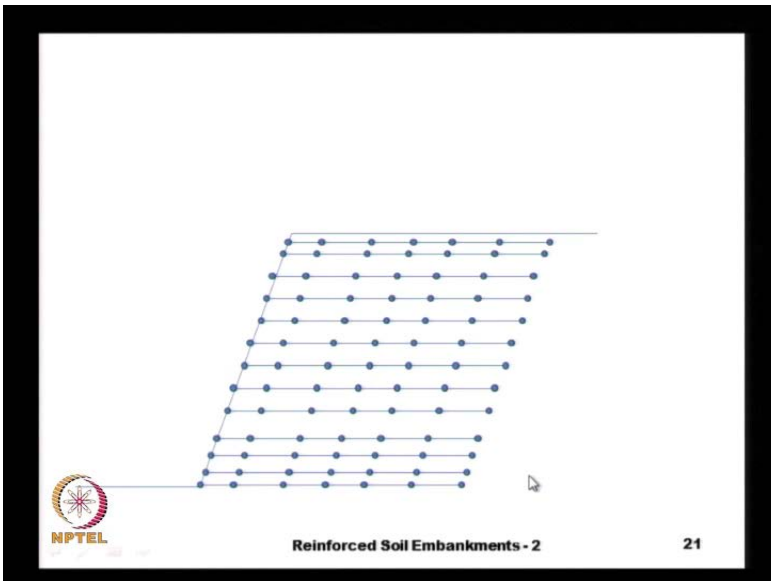

So, now we need to check whether this 13 meters length is adequate from from the other considerations, and this is the schematic of the different reinforcement layers. So, of totally 16 layers provided that different depths as as we have indicated here.

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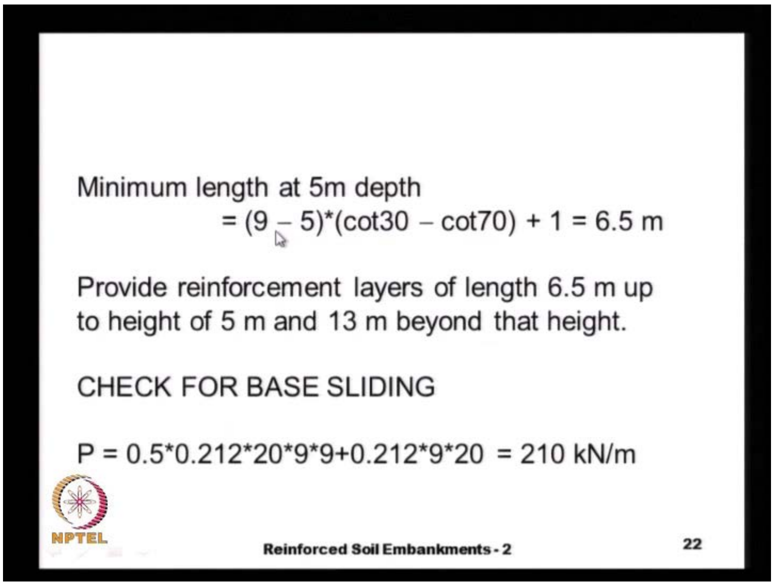

And let us calculate the what is the minimum length required at some other depths. Let us say that at 5 meters depth somewhere in between in in mid way through it is only 6.5 meters. So, now, let us aim for lower length of 6.5 meters in the bottom portion because in the in the bottom portion our a length of embedment is going to be very very high if you provide full 13 meters, and we can try it economize by providing a shorter length of reinforcement in the bottom portion.

So, now the provided reinforcement layer length is 6.5 meters up to a height of 5 meters and 13 meters beyond that height. Let us let us just have a very simple design only two lengths 6.5 meters in the bottom portion and 13 meters at the at the top of the the slope. Let us now check for the base sliding and the total lateral force is one half K a gamma H square H is 9 meters plus K a times H times q, that is 210 kilo newtons per meter.

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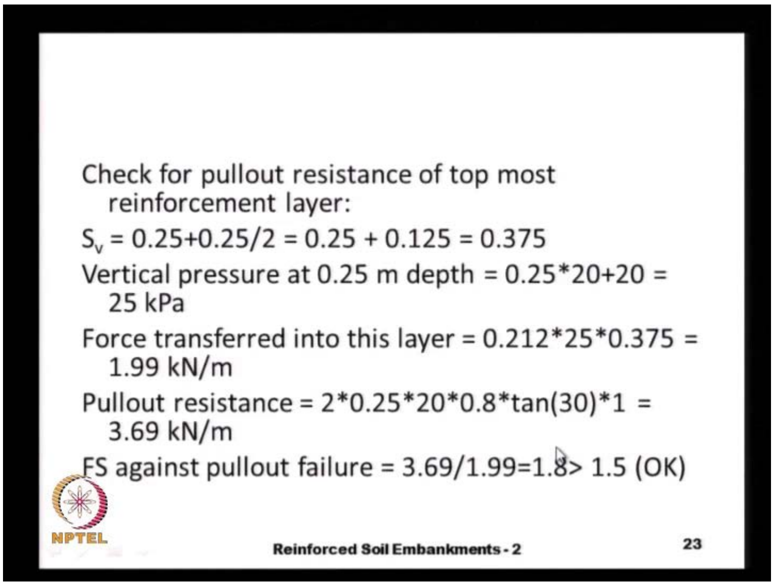

And the oops.

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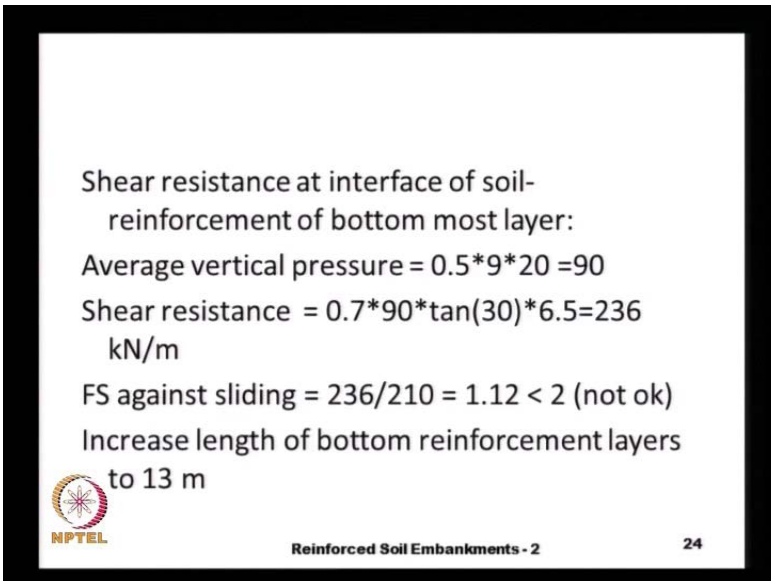

The shear resistance at the interface of the soil to reinforcement layer is average vertical pressure is 0.5 times nine times 20 that is 90 k P a. And the shear resistance the our a shear interaction parameter is is 0.7 0.7 times vertical pressure is 90 times tan phi that is tan 30 times 6.5 that is 236 kilo newtons per meter, and the factor of safety against sliding is 236 divided by 210. And this factor of safety oops this factor of safety is coming as 1.12 which is less than 2. So, it is not sufficient. So, we need to increase the length of the reinforcement.

So, we need to increase the the length of the bottom reinforcement layers to 13 meters. So, we can provide uniform length of 13 meters of the full height of the the slope and now we need to also check for for the the pullout. And then the rapture let us check the and the top most reinforcement layer and the the the top most reinforcement layer takes the force that is coming from the soil above this reinforcement layer, and on the force over the half spacing that to the next bottom layer.

So, it is the S v is 0.25, that is the the height of soil above the above that particular layer plus 0.25 by 2 that is 0.375. And the vertical pressure 0.25 meter depth is 0.25 times gamma plus q that is 25 k P a. And the force transferred into this layer is K a times q times S v 0.375 that is 1.99, and the pullout resistance, because at the this height we have only 1 meter embedment length. So, it is two times 0.25 times 0.25 is the height of soil times 20 as here, I am ignoring the effect of surcharge.

Because as per the codal requirements we should ignore the effect of live loads surcharge on on the pullout resistance, because that is the worst case scenario is actually that is also as per the case b of the b s 8006. And the pullout interaction parameter is 0.8 and this two is because we have the shear force developed on both the surfaces times tan 30 times one is the embedment length; that is 3.69 kilo newtons per meter. So, the factor of safety against pullout failure is is greater than is greater than 1.5 it is its coming as 1.8. So, it is safe.

So, our the length of the reinforcement layer is 13 meters that is sufficient as far as the pullout and the base sliding is concerned, and the vertical spacings are as as designed earlier and because our vertical spacing that is provided is much lower than the permissible spacings. We will not have any rapture rapture failure and the. So, far we have discussed the stability of slopes under static loading, but in many cases will have to design the slopes even for earthquake forces.

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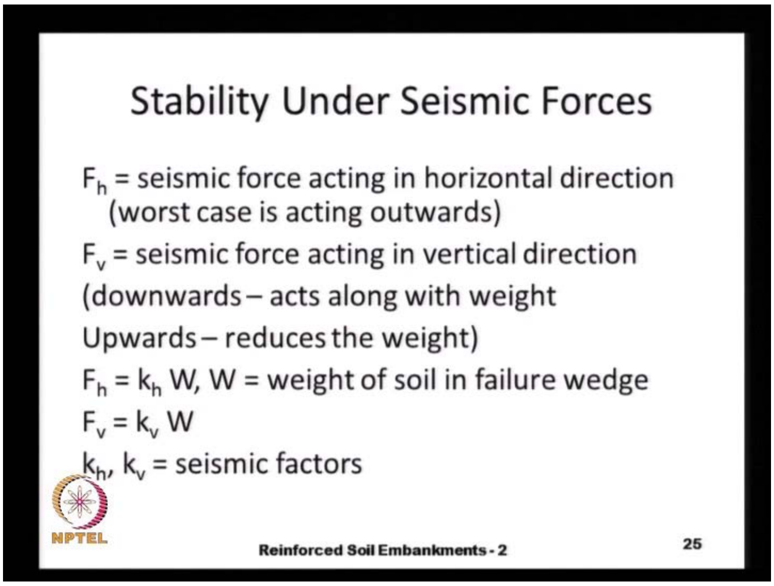

And now let us include the additional force that we have, because of the seismic seismic forces and let us say that we have two extra forces F h; that is the seismic force acting in the horizontal direction and this horizontal force, because of the seismic action could act into the soil are away from this soil. And in the worst case is the F s acting away from the soil, and because if it is acting into the soil it, it tends to stabilize the soil, and F v is the seismic force acting in the vertical direction. And it could act either downward or upward, because it can act both ways. And if it act downwards it acts along with the weight of the soil. So, it increase the normal resistance, but then also increases the the shear force and if it acts upwards it reduces the weight. So, it reduce not only the normal force. But also the shear shear force and f h it can be written as sum factor K h times w where W is the weight of the soil within the failure wedge and F v is the K v times W where K h and K v are the seismic factors. And there are so many empirical factors given to estimate the K h and K v values and as we have discussed earlier.

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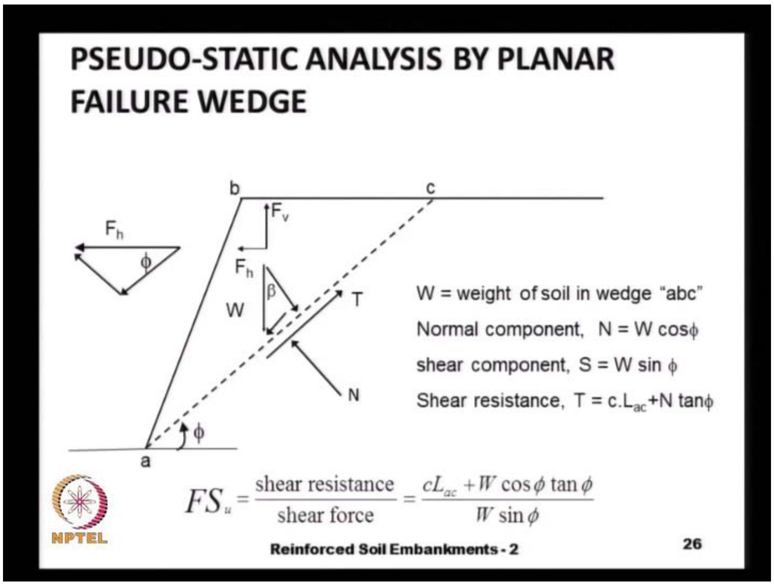

We need to use the earliest 1893 for estimating these factors . So, now let us look at the pseudo-static analysis of this slope the subjected to seismic action. Let us consider the same slope at an angle of beta, and the rapture surface is is is at an angle of phi that is the the the most critical rapture surface and we have are layer considered the normal force n t and then the shear force acting down. And now we need to consider two additional forces F h and F v and F h is the horizontal seismic force that is the most critical one if it is acting out outwards, and F v can act either upper down and the factor of safety of the unreinforced slope is is like this. As we have seen earlier and by considering the by considering the action of this additional forces. We need to consider the or we need to determine.

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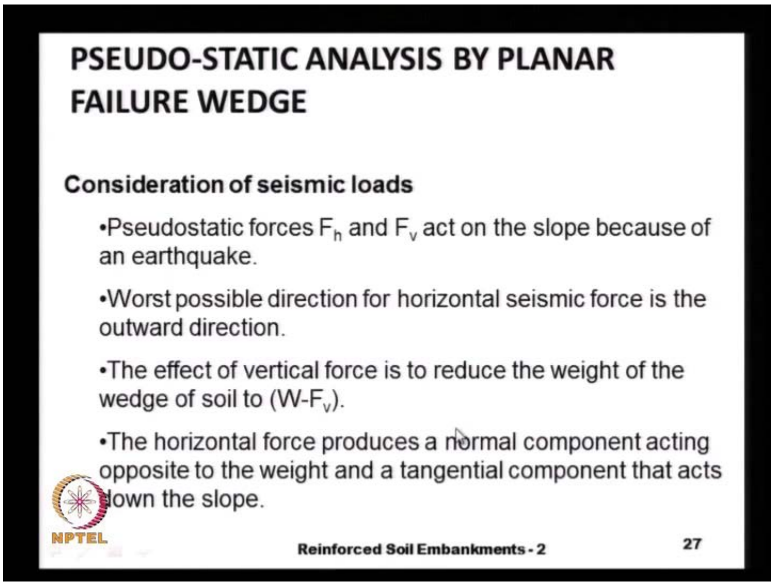

The factor of safety of the slope the most critical case and our the pseudo-static forces are the F h and F v. And the worst possible direction for the horizontal seismic force is the outward direction, where as the the vertical load vertical force, because of the because of the seismic excitation is either to increase or decrease the effect of the weight.

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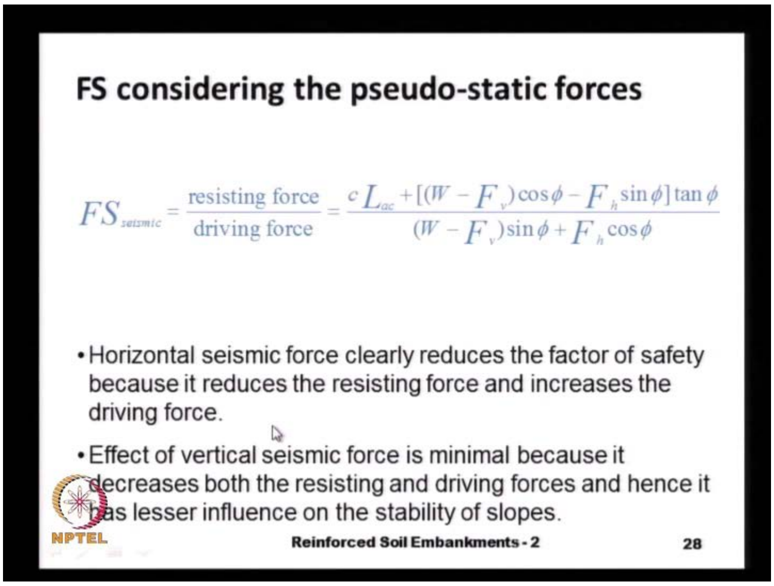

And if you resolve all the forces our resisting force is C times L a and W minus F v times cosine phi assumed that the vertical force is acting up and minus F h sin phi. This whole thing times tan phi. So, you see that the effect of the horizontal seismic force is to reduce the resisting force, it is reducing the normal force; that is acting and this whole thing divided by the driving force that is the W minus F v. That is the vertical force plus F h cosine phi and F h cosine phi is the is the the shear force acting on the slope because this horizontal component. So, this the horizontal seismic force reduces the factor of safety because it is reducing the resisting force while increasing the driving force because it is reducing the a numerator.

And increase in the denominator where as the effect of the vertical seismic force is the same like it is decreasing the the a numerator, and is also increase decreasing the denominator. And if we assume that the vertical force is acting down it becomes w plus F v and the numerator and W plus F v in the denominator. So, it is the effect of F v is a very minimal whereas, effect of F h is quite significant.

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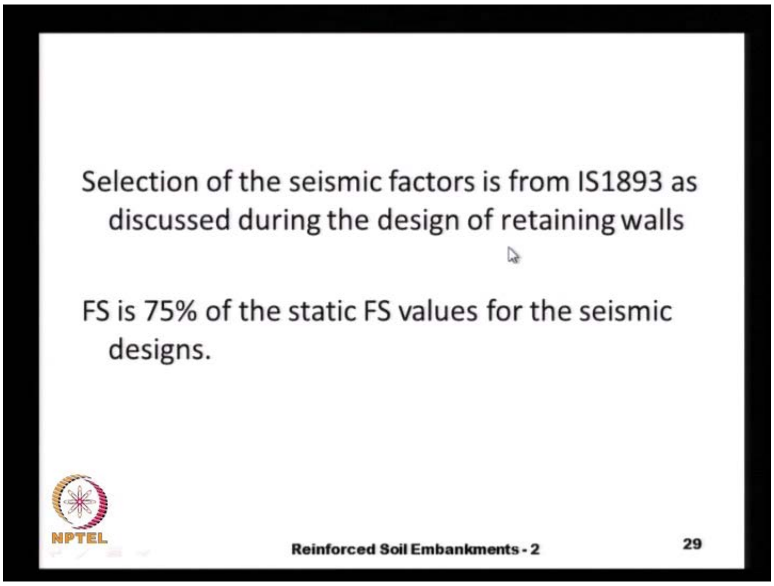

And how do we select the K h and K v once again we can go back to our IS 1893 the the code of practice for design of the seismic resistance of the buildings. And this we have already seen how to apply for the design of retaining walls the same principles apply even for this case. And when we consider the seismic forces our factors of safety can be reduce by reduced to 75 percent and the static values as we discussed earlier say for example, if we need factor of safety of 1.5 and the static is under seismic case it should be it can be 1.125.

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So, that is the end of this lecture, and if you have any questions you can contact me at this email address.

Thank you.