Geosynthetics and Reinforced Soil Structures
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Lecture - 12
External Stability Analysis of Reinforced Soil Retaining Walls

Good morning students. In the previous class, we have been discussing about the design of reinforced soil retaining walls, and let us continue the discussion in this class.

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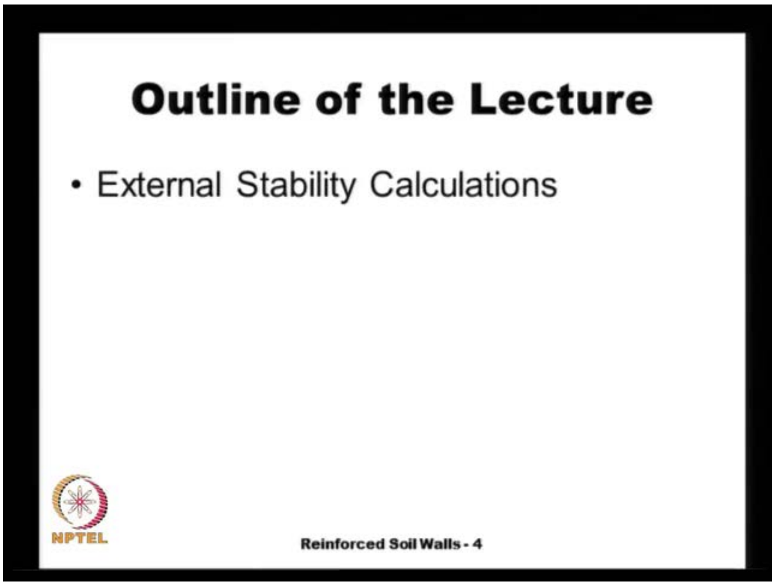

The outline for this lecture is will completely discuss about the different external stability analysis.

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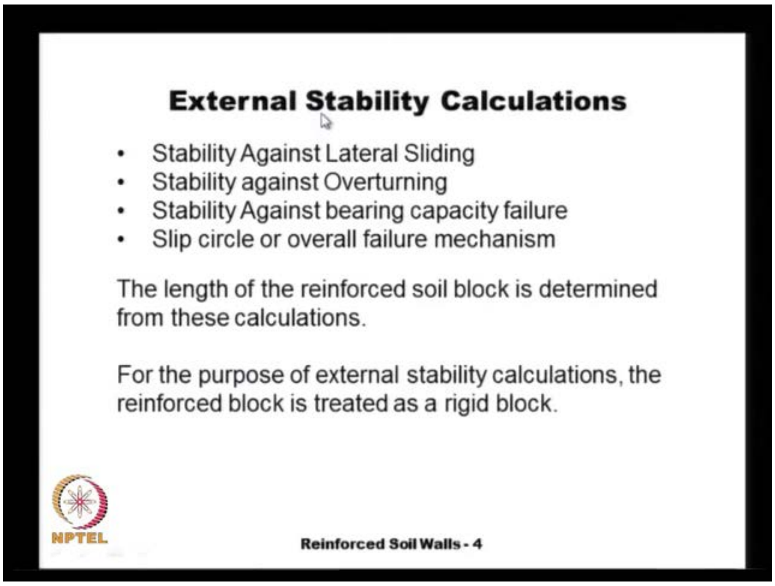

Just give a brief outline as we discussed in the previous lecture, there are a different calculations we need to do to satisfy the external stability conditions. The first one is the stability against lateral sliding, because our retain wall is retaining the soil at an angle stepper than it is shown the angular free pose there is tremendous lateral pressure that is acting on the wall.

And it tries to, the soil tries to push the entire reinforced soil, and our retain system retained soil system should have adequate fact of safety against lateral sliding. And it should also not over turn that is it should not top lower, because of the moment that is applied by the external forces, and then there should adequate fact of safety against bearing capacity failure.

And there should not be any slip circle type failure, especially in the case of walls constructed on in steep slopes or on extremely soft foundations soils, and for our calculation purposes we treat the entire block of the soil as a homogenous block, and as a rigid block. And consider the affect of all the forces that are acting on this block.

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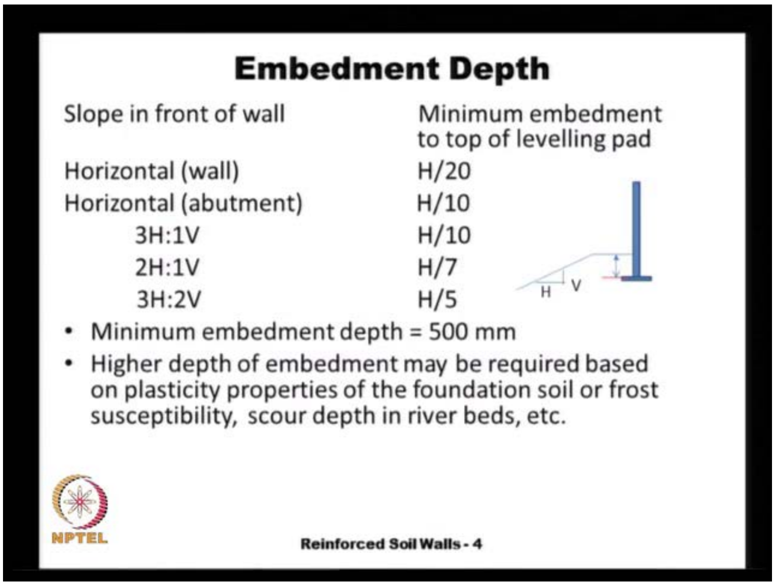

And when we construct the retaining walls we always give some embedment depth, and as per the federal highway administration code this embedment depth is measured from the level of the of the soil up to the top of the leveling pad shown here. And that depth depends on the slope angle in front of the wall, and for a horizontal fill our embedment depth is h by 20 and for and if it is an abetment it is h by 10.

The bridge abetments in variably they we have vary large lateral forces acting on the on top of the retaining walls, and because of that we give a higher embedment depth to increase the stability of these retaining walls. And so because of that if it is an ordinary retaining wall the embedment depth is h by 20, and if it is a bridge abatement it is h by 10, and depending on the slope angle the embedment depth may also change.

For the most critical case the embedment depth may be almost 20 percent of the wall height h by 5, and the minimum embedment depth is given as 500 millimeters. They assume a recall from the construction of shallow foundations, this 500 millimeter is the same as what we have even for the shallow foundations. And this is these are just generic guidelines, but depending on the site conditions these embedment depths may be higher. Especially a when you have foundations soils with highly plastic properties, that is the foundations soils that may have undergo lot of volume changes, either expansion during the rainy session or contraction during the summer session.

Our foundation of the leveling part should be a starting below the depth of soil, that is subjected to seasonal moisture changes, that leads to a volume changes. So, that we have better stability or in the case of cold regions, where the top soil is subjected to frost action. That is when it is during cold season the top soil freezes and in the process it expands, because when the water is flow is in it expands in volume or if your retaining wall is in the along river beds and so on. The wall should be taken below the maximum cover depth that we estimate, but in normal case minimum embedment depth is 500 mm or one of these which ever one is higher.

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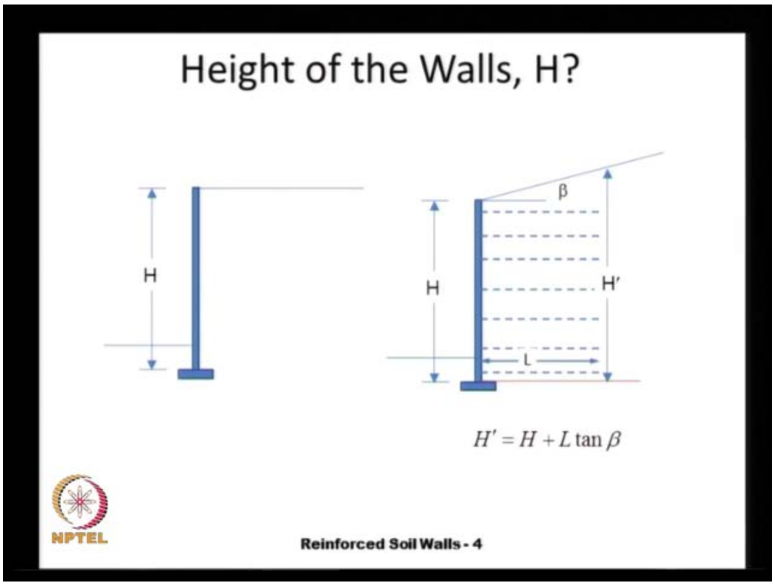

So, for the design purposes we need to calculate the height of the retaining wall, and the height of the retaining wall is defined like this, when you have a horizontal back fill. The height of the wall is always calculated as the height measured from the top of the leveling pad up to the top of the back fill soil, this is the height and we neglect the soil that is their in front of the wall, because that is liable to escalated for maintenance purpose and so on.

And for this soils with back fill slope of beta the height is defined, once again from the top of the footing and the in terms of the tan beta, our modified wall height is defined as h that is the front face height plus l times tan beta. So, these walls they will have a slightly higher lateral forced to be supported, and in terms of the design of walls with horizontal back fill is more easier, because the height is constant.

Where as the walls with inclined back fill the height of the soil to be considered depends on the length, so it goes through like we need to go through some alterations, because if you increase the length of the reinforcement. To increase the stability your height of the wall increases there by our lateral forces, and they over turning or means they increase, so it is an alternative process that we will see in the examples that we work out later.

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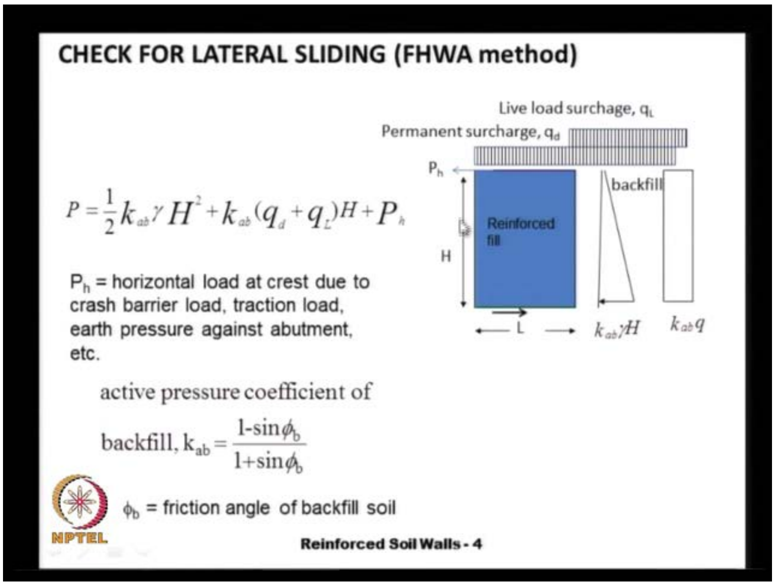

So, now let us see how we can check for the lateral stability of the wall, and the federal highway administration method is the simpler one, where as the BS code method the British code 8006 is a bit more complicated, but ultimately both of them are equivalent. Let us consider a reinforced soil wall like this, having a block length of l and height of h and for simplicity lets you consider only horizontal back fill, because the principles are the same.

Whether, have a horizontal back fill or inclined back fill and lets say that there is permanent surcharge q d that may be, because of the weight of the soil that is resting on top of our reinforced wall. And then there could be a live load surcharge the q l, and when it comes to the live load surcharge we only consider the live load for the purpose of excitation, that is the forces that cause destabilization. And we neglect the effect of live loads when we calculate the resistance forces, so because of that I have purposely shown in the live load like this, and actually this scenario that is proposed by federal highway administration corresponds to case b the load case b of the BS code.

And let us say that there is a horizontal force of P H acting at the crest of the wall and this could be either because of the traction forces that are there because of the braking and other loads or it could be because of crash barrier load. Because, of vehicle impacting on a on a crash barrier or it could be because of the pressures that are exited on bridge abetments that is directly resting on top of the retained soil fill.

And so the effect of the self weight is to have a triangular pressure distribution like this, and the effect of live load surcharge or the permanent surcharge the dead load surcharge is a rectangular one like this, and the lateral load is one half k a b. And once again theoretically we have two types of fills one is the soil fill, that is placed directly inside the reinforced block and the another soil fill that is placed behind the reinforced block, and as we have discussed earlier the soil that is used in the reinforced block should be highly granular.

So, that there is a good interaction between the reinforcement on the soil whereas, the back fill soil, it could any thing it could even a marginal soil, but then it should meet some specific requirement as per the codal provisions. And let us say that the reinforced soil a s some properties and then the back fill soil has some other property, and this k a b refers to the active earth pressure coefficient of the back fill soil.

And similarly, the k a or refers to the active earth pressure constant within the reinforced fill, and the lateral forces one have k a b gamma h square. Where, h is height of the retaining wall plus k a b times q d that is the dead load plus q l that is the live load surcharge times the height of the wall plus the p h, and the k a b is 1 minus sin phi b by 1 plus sin phi b for a simple case.

And if you have a inclined back fill or a non vertical retaining wall, we can use the coulomb's equation more general equation and once you calculate the excitation force are the lateral force, we need to calculate what is the resistance force that is acting below the reinforced fill and so that we can calculate the resistance.

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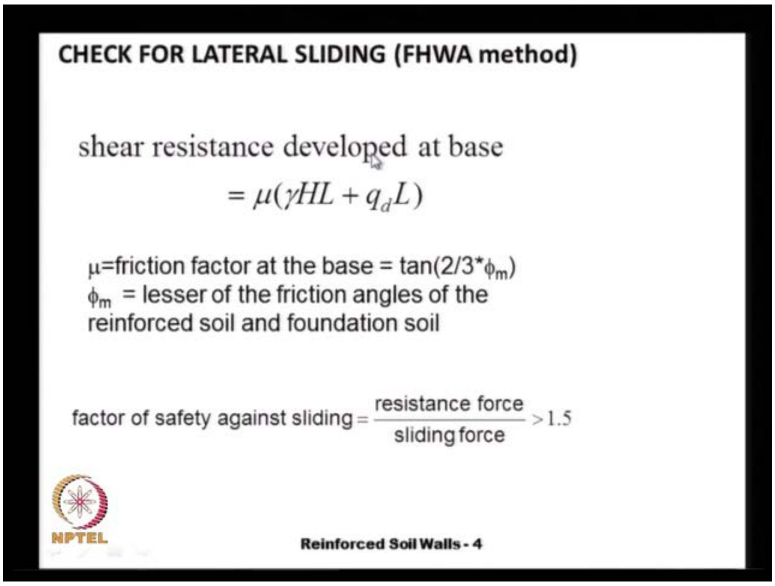

And shear resistance that is developed at the base is the mu that is the friction factor that is developed at the base of the retaining wall multiplied by gamma h l.

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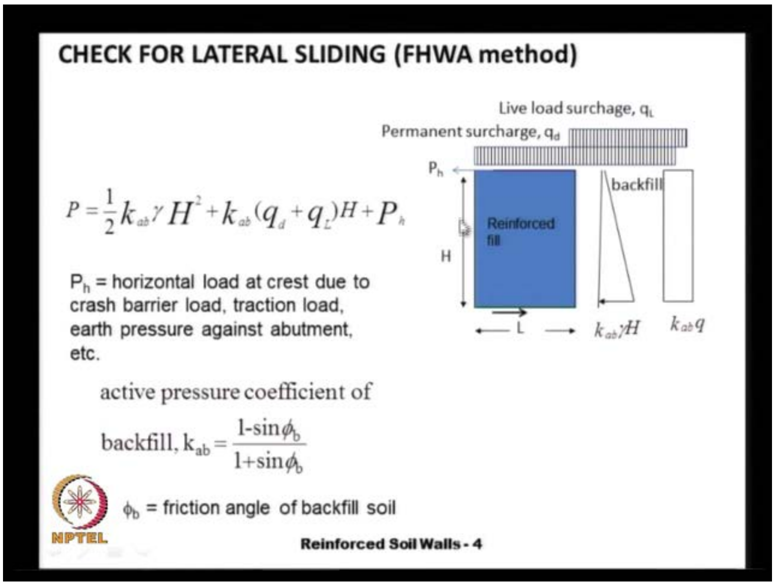

That is gamma h l is the gamma is the unit weight of the retained soil fill multiplied by height and l and all the calculations are done per unit length perpendicular to the plane of the analysis, just as how we did in the regular geotechnique engineering courses either the design of the retaining walls or the design of slopes and so on.

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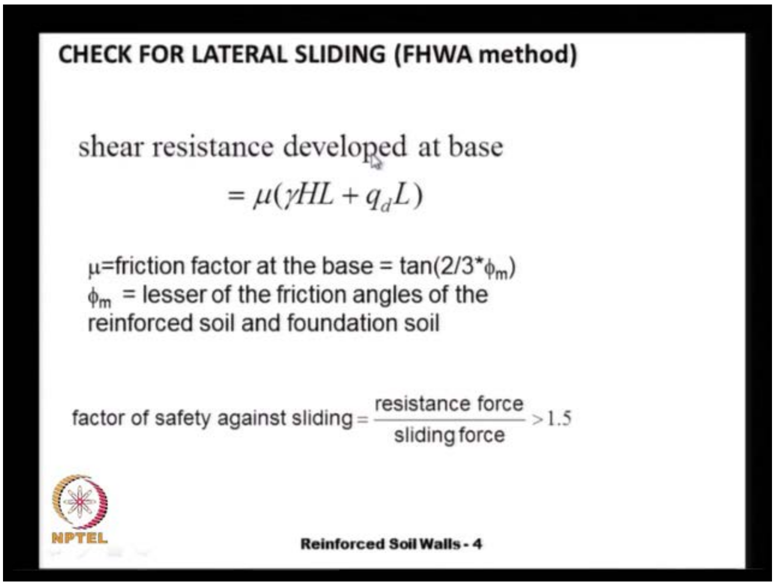

So, our gamma h l is the weight of the soil within the reinforced block plus the q d that is the permanent surcharge multiplied by the length l multiplied by one unit length in the perpendicular direction will give us the magnitude of the preeminent load. That multiplied by mu is the frication factor that is acting at the base, and in case of any lack of data of the foundation soil.

We just simply take mu as tan of two thirds phi m, where phi m is the is the friction angle the lesser of the friction angle of the reinforced fill and the foundation soil. And in some course they also write it as two thirds of tan phi both go together like depending on the foundation code, and depending on the codes, for the reinforced soil retaining walls there is slight variation.

And then the factor of safety against sliding is resistance force divide by sliding force, and the resistance force is this quantity divide by the sliding force. And this should be greater than 1.5 in the federal highway administration code, this should be greater than 1.5 whereas, in the biggest code is actually all the excitation loads are multiplied by with a factor of 1.5.

Whereas, the resistance loads they are factored with a factor of 1, so there also we get the same 1.5 factor of safety, when we satisfy the equilibrium between the resistance force and the excitation force. And what if this factor of safety comes out as less than 1.5, we just have to increase the resistance force and the only way to increase the resistance force is by increasing the length of the reinforcement block the l as you can see or we can use a high better quality soil fill.

So, that you get a higher friction factor, but among these two increasing the length of the reinforce block is a bit more simpler, because there may be limitation on the type of soil that you can bring to the site. So, in variably we only look at the option of increasing the reinforced block length for achieving higher stability.

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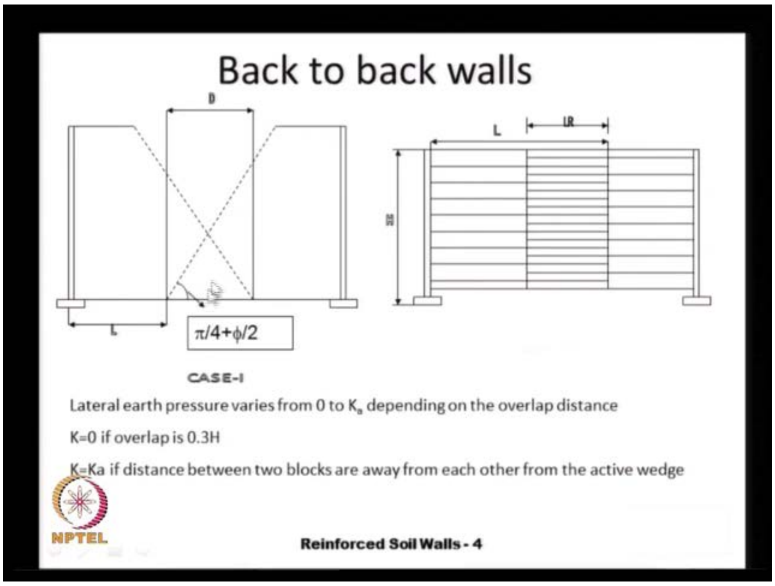

There could be a some special cases that arise during the construction and once such case is the back to back walls as we have seen, we may have a two walls constructed on both sides of an approach road, and if they are closed together then the earth pressures that act on the walls could be different. So, actually if there is an overlap of the reinforcements like this, because these two retaining walls there close together our earth pressures could be depending on the amount of overlap, they could be 0.

See if the overlap is a h grater than 0.3 times the wall height the federal highway administration says that, the k could be 0; that means, that we do not have to check for lateral sliding stability. We only check for overturning or the other types of failures, and if k a if the blocks are away from each other like this. Where, let us say that the Randkin active rupture surface that is drawn at an angle of pi by 4 plus phi by 2, if it does not intercept the reinforced block of the other side wall we assume that k is equal to k a and in between if the overlap length is less than 0.3 h we just do a linear intercalation.

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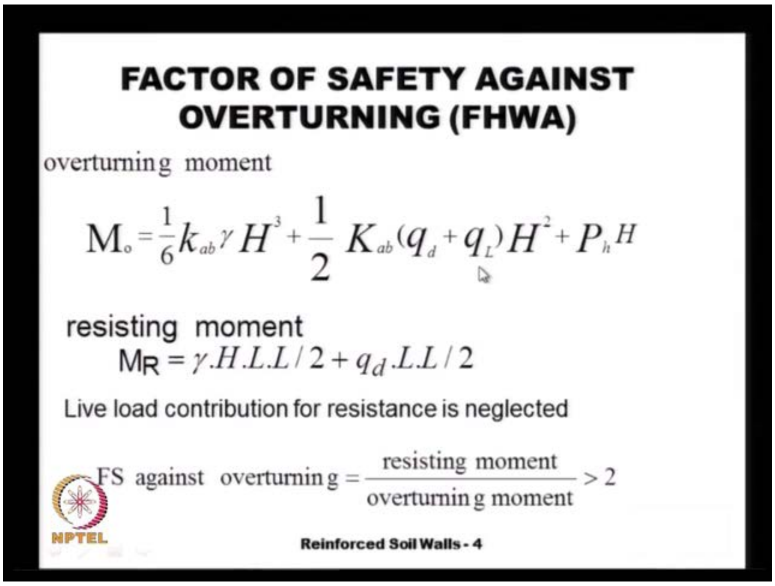

And similarly we need to look at the stability against over turning, against the toppling over, and once again the FHWA procedure is discussed here, because it is a simpler one, and the BS code methods we will discuss later on.

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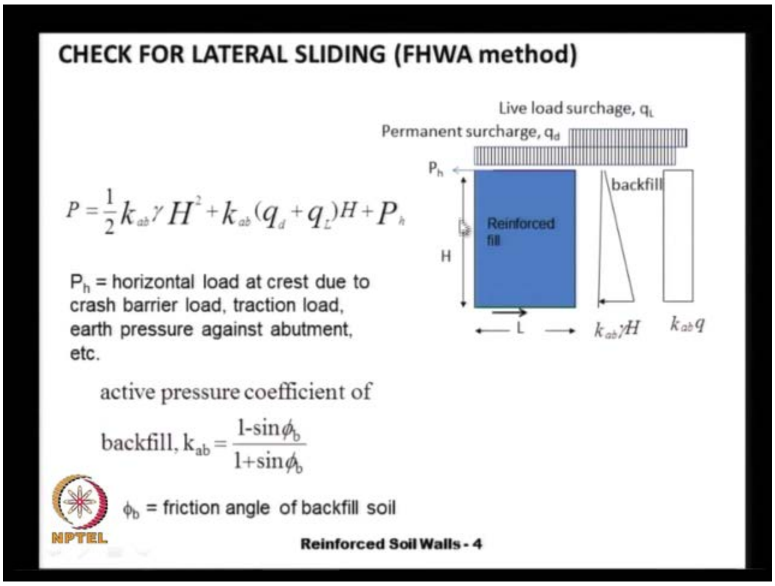

The overturning moment is once again, because of the triangular distribution and the rectangular distribution and we realize that the center of gravity of the triangular is that one third of the height whereas, the rectangle is one half of the height.

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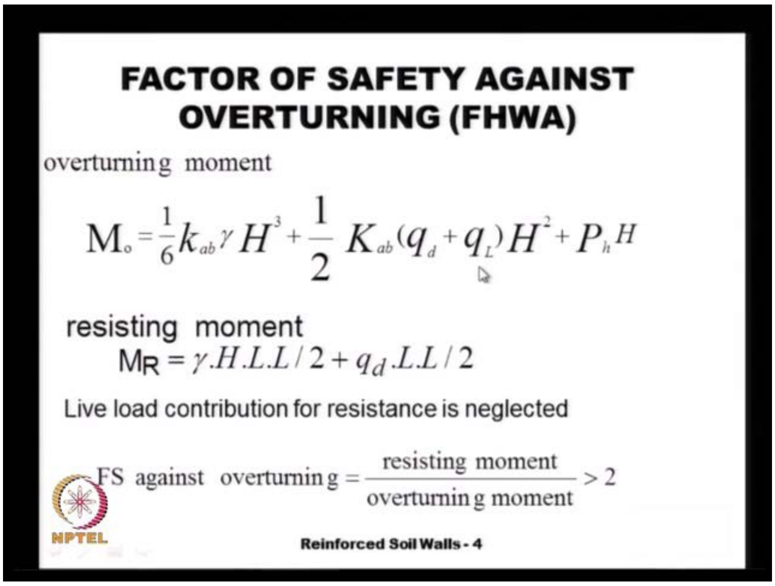

So, so if we take moments about the base the overturning moment is one sixth of k a b gamma h cube plus one half of k a b q d plus q l, that is the dead load surcharge, the live load surcharge times h square plus P H times the wall height. P H is the is the traction force or the lateral force that is acting at the at the crest of the retaining wall, and the resentencing moments the overturning moment considers both the dead load surcharge, and the live load surcharge. Whereas, the resentencing moment it considers only the dead load surcharge and not the live load surcharge, the lets go back to the this figure once again.

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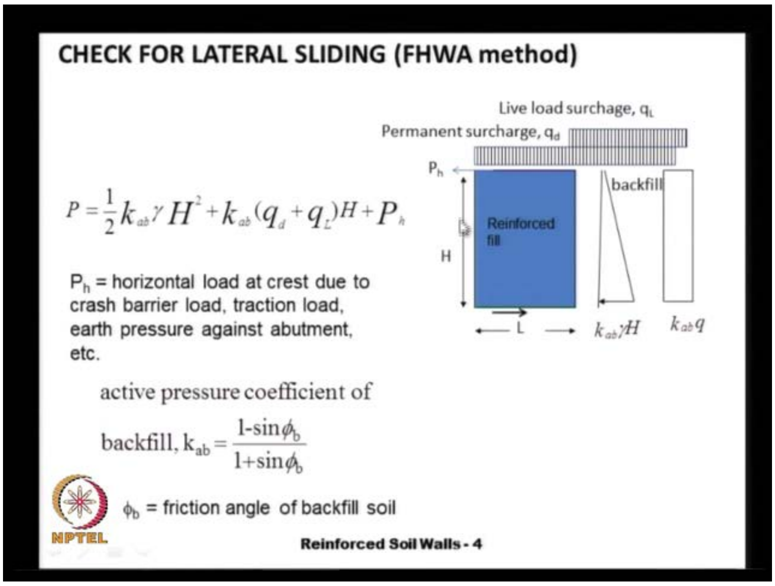

The resentencing moment is acting in this direction and the weight of this reinforced fill multiplied by l by 2, and the load the permanent surcharge load that is acting on the surface multiplied by this l by 2.

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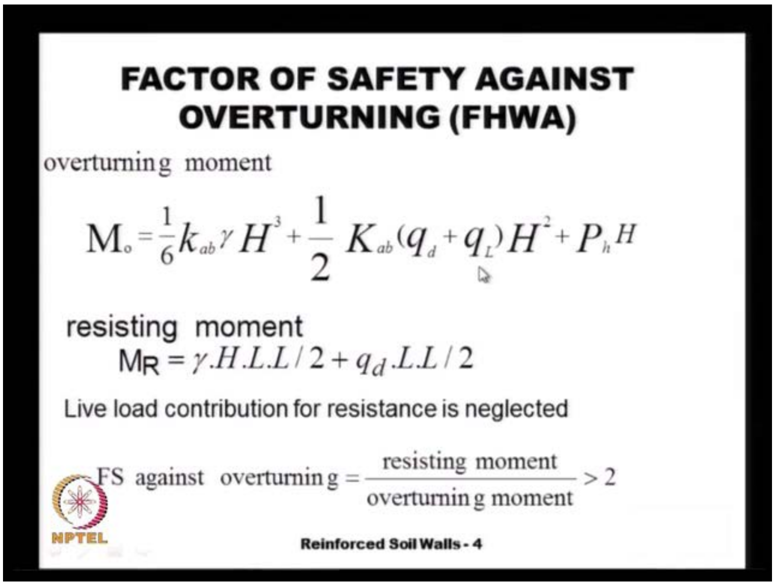

And as I mentioned earlier we neglect the live load contribution for the stability of this retaining wall, and the factor of safety against the overturning is resisting moment divide by the overturning moment. And it should be greater than 2 at the minimum and in some cases our factor of safety may be more than 3 depending on the case that we have.

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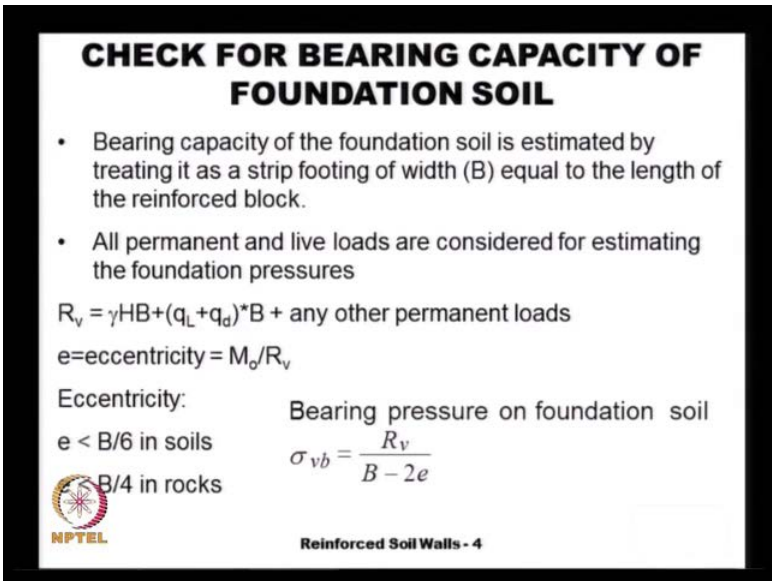

And the other condition that we need to satisfy is the bearing pressure or the bearing capacity of the foundation soil, and the bearing pressure is calculated by distributing the load at the base. And we used the mayor halves formula for calculating the bearing pressure in the federal highway administration in a code whereas, in the BS code method directly the pressure that is acting is taken as the bearing pressure. And the total load downward load that is acting including both live load surcharge and the dead load surcharge is like this.

The reinforced block is gamma h times b plus because of the surcharge q l plus q d times B plus there could be some other permanent loads are because of the weight of the bridge abetment, and the loading that is acting we need to consider all those loads. And that is our the total vertical load of the reinforced block, and our eccentricity e is M naught, M naught is the overturning moment divide by R v, where R v is the total vertical load, and this eccentricity e should be less than B by 6 where B is the length of the reinforcement layers B by 6 in the soils.

And in the case of reinforced soil retaining walls directly constructed on top of rock this e could be less than B by 4, it could be the e could be slightly higher case of very stiff foundations soils, this is similar to the I R C codal provisions for design of retaining wall the rocky straighter. And our the bearing pressure that is acting on the soil is R v that is total net load that is acting on the foundation soil divide by B minus 2 e, B minus 2 e is our effective length over widths the foundation pressures acting, because this k amount of the mayor halves finding.

Where, in when there is a any eccentric loading the footing looses contact with the soil over length of 2 e, where e is the eccentricity and effectively the footing is supporting load over a length of B minus 2 e. So, our bearing pressure is R v by B minus 2 e, and this bearing pressure should be within certain allowable limits, and that is called as the allowable bearing pressure.

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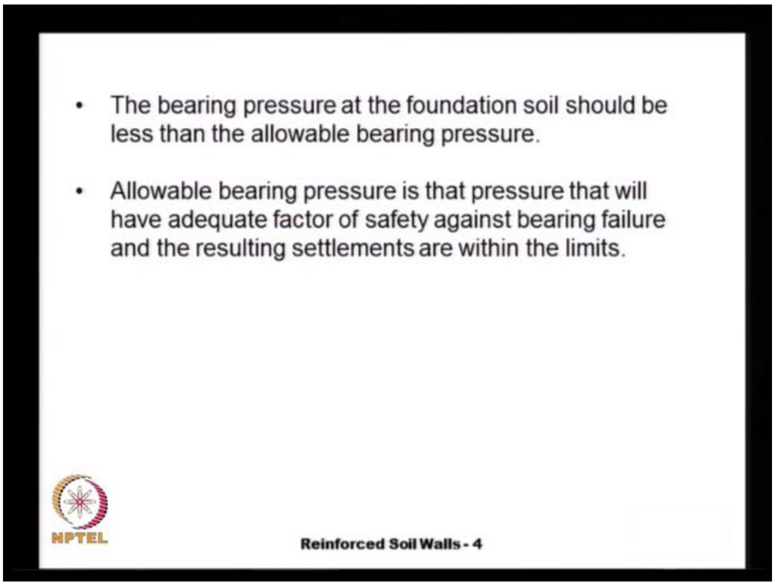

And the allowable bearing pressure itself is defined as that pressure, that we can apply that will not cause any foundation failure due to the bearing capacity exceed, the bearing pressure exceeding the bearing capacity by certain factor are the settlements exceeding our permissible limits, so that is called as the allowable bearing pressure.

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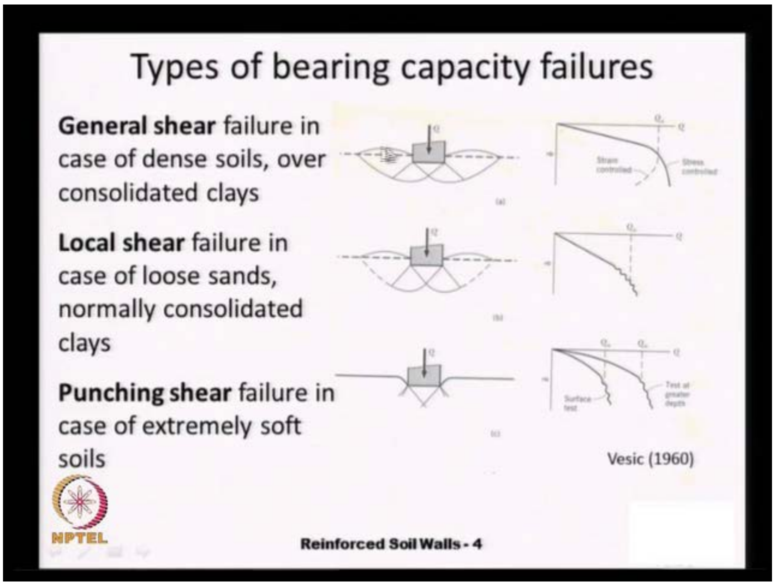

And let us see how we can calculate the allowable bearing pressure of the foundation soil, if you may recall we have 3 modes of bearing capacity failures. One is the general shear failure that happen in the case of dense sands or over consolidated clays, where in under the application of footing loads. There is a very clear development of the rapture surface in the foundation soil, and there is good amount of soil heaving and the pressure settlement graph it has a shear certain sharp peak.

And the other extreme is the punching shear failure that happens in the extremely soft foundation soils, when you apply some pressure till just simply sinking without development of any rapture surface. And in between we have the local shear failure that happens in the case of loose sands or normally consolidated clays, and in the case of local shear failure the rapture surface is well developed only below the footing, and away from the footing it is partially developed.

And the soil heaving is also not as high as in the case of general shear failure, it may or may not happen. And the pressure settlement data for punching shear failure will show a very sudden increase in the settlements, whereas in the local shear failures there is significant increase in the pressure and then after that there is failure.

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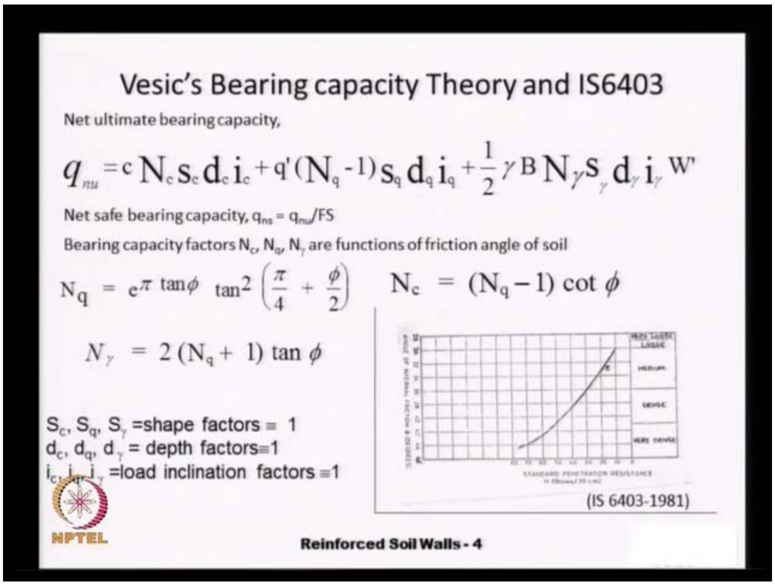

And the bearing capacity is estimated as per the I S 64 0 3, that is applicable for bearing capacity of shallow foundation, and we treat the reinforced soil retaining wall as a strip footing having the width equal to the length of the reinforcement. And by you using this formula that is the net ultimate bearing capacity q and u is c N c plus s c d c i plus c plus q prime, that is the over burden pressure effective over burden pressure at the footing level times n q minus 1 times s q d q i q plus 1 half gamma b and gamma s gamma d gamma a gamma w prime.

This entire formula gives us the net ultimate bearing capacity and the net safe bearing capacity q n s is obtained as q n u by factor of safety, and usually the factor of safety that we use for design of reinforced soil retaining walls is any where from 2 to 2.5. And this value that we use for factor safety it depends and how much confidence we have in the foundation soil properties that we have used for estimating our bearing capacity.

In this equation our c is the cohesive strength of the foundation soil N c N q and N gamma they are the bearing capacity factors, which are written in terms of the friction angle as shown here. These are all given by basic in his paper that was published in 1973 N q is given by this N c is given by this and N gamma is given by this formula. And the s c s q and s gamma they are the shear factors and because the original bearing capacity theory was developed for a strip fitting the and our retaining wall is treated as a strip fitting, we can take all the shear factors as equal to 1.

And the depth factors d c d q d gamma, they are actually written in terms of the friction angle of the soil, but in this case in the case of reinforced soil retaining walls these are usually constructed as shallow depths may be at about half a meter depth. And compared to the width of this the reinforcement that we have this depth may be, so little that in variably the depth factors will come out as very near to 1. So, we can take them as one in case where your the reinforced block is very shallow and very small width, that is of the order 2 meters.

And your embedment depth is one meter, and in that depth ratio of 0.5 is quite significant, in that case we can use the relevant formula for estimating the depth factors. And our i c i q and i gamma they are load inclination factors and our lateral loads that act on these retaining walls is they are, so small compared to the vertical loads, that we can just simply take them as one all these factors.

In case there is a very significant lateral load then we may have to apply some correction factors is i c i q and i gamma, they are less than or equal to 1, and how do we estimate the friction angle phi the I S 6403. They have even a small chart in terms of the corrected standard penetration value on the x axis, and the friction angle phi on the y axis we can use any of these empherical charts to estimate our friction angle.

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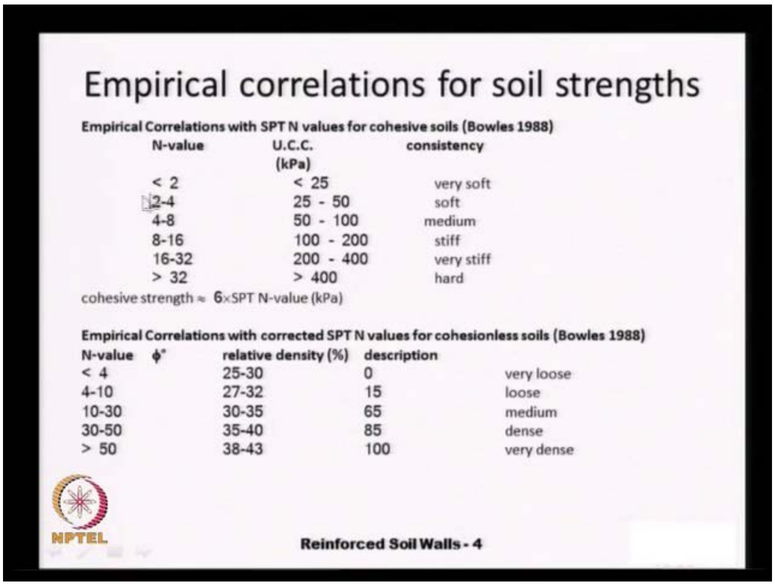

These are some tables that have obtained from different text books, the first one is from Bowles, this is given for cohesive soils in terms of the SPTN value, as you know the SPT is the Standard Penetration Test. That we commonly perform in the geo technical engineering and SPTN value is less than 2 the foundation soil is very soft, and 2 to 4 is soft, 4 to 8 is medium and so on, and the different ranges of SPT values will have different ranges of the unconfined compressive strength.

And the once you have the UCC the CU is one half of the UCC, and the absence of this stable, we can take the cohesive strength as approximately equal to 6 times the SPTN values in SI units that is the kilo Pascal’s. And a similar correlations for sands is given like this depending on the value of n, the friction angle phi could vary any where form about 25 to 43. And the n value less than 4 signifies very loose sand deposit and n value greater than 50, it signifies very dens sand and these are the relative density values any where from 0 to 100 percent.

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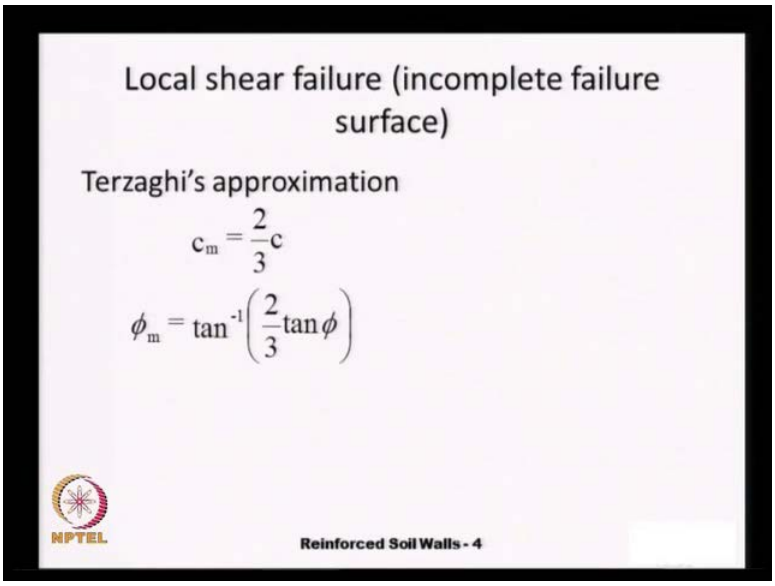

And when we have a local bearing failure, local shear failure of the above formulas that we have these are not valid manly, because those formulas are derived for a case where our rapture surface is completely developed.

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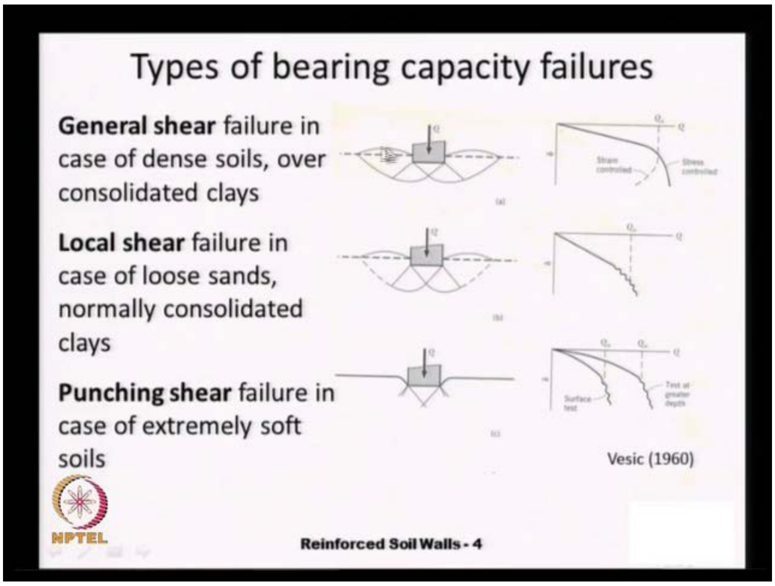

As in the case of general shear failure, and when we have the local shear failure there is we do not know what is this rapture surface, although we just speculated that it will follow this curve and meet the surface, but we do not know.

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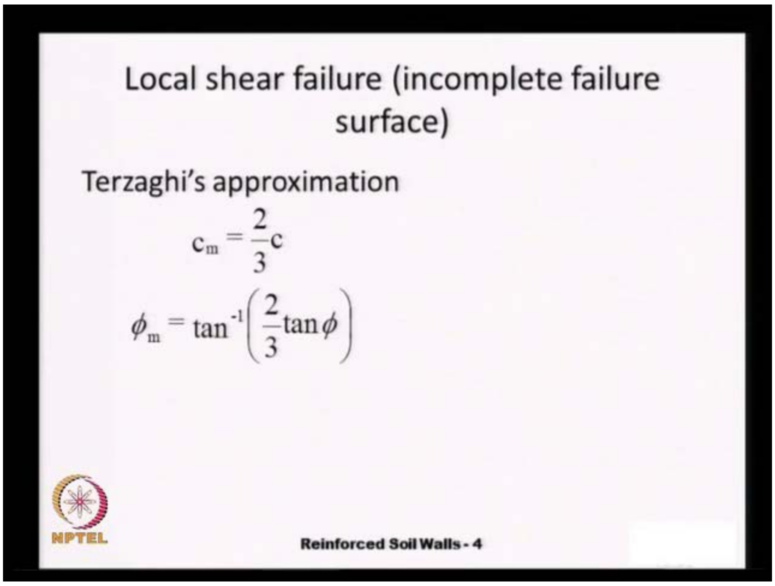

And Terzaghi being very highly practical person, he suggested that the shear strength properties can be slightly adjusted the account for local shear failure, and he suggested that the modified cohesive strength is two thirds of c, and the phi modified tan inverse of two third tan phi.

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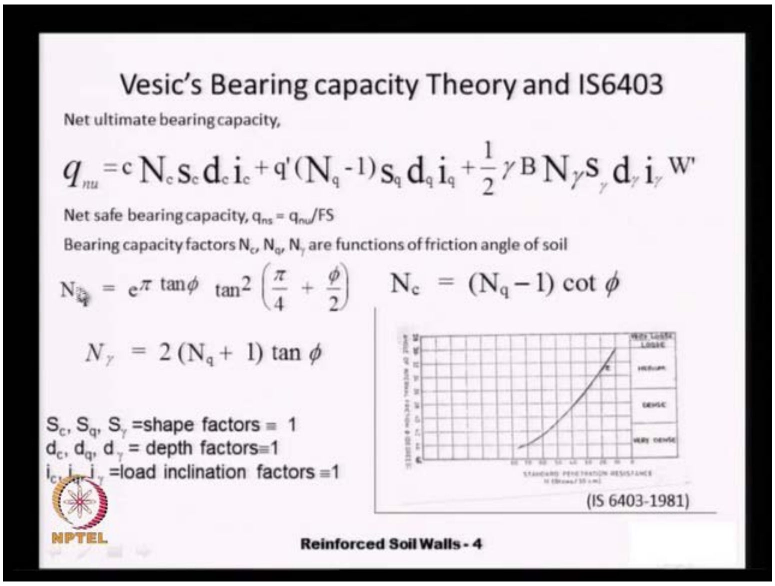

And once you have the c and m phi m we can go back and substitute the phi m in place of phi to calculate our bearing capacity factors N q, N c and N gamma and c m in place of c here.

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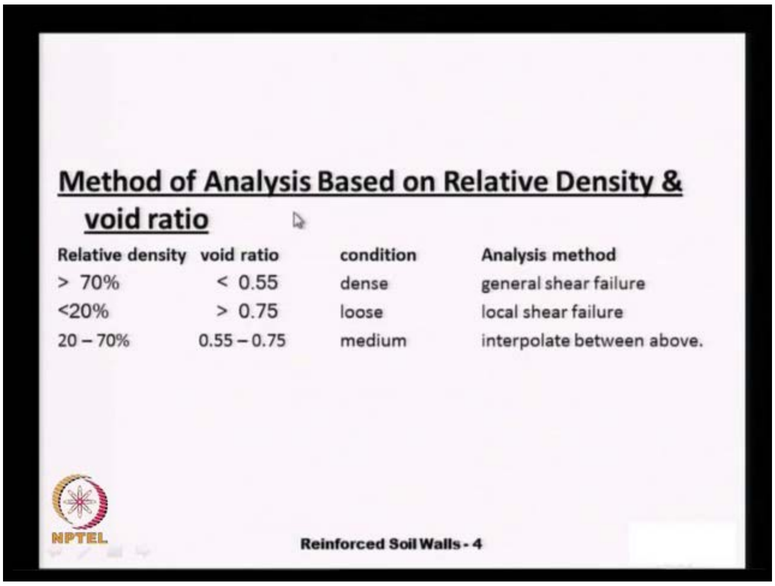

And depending on the relative density that we have, we may have either general shear failure or local shear failure, and from the I S 6403, we get that if the relative density of the soil is greater than 70 percent or if the void ratio is less than 0.55. We can have a general shear failure for the under the bearing loads, and if the relative density is less than 20 percent or if the void ratio is greater than 0.75, we may have local shear failure. And in between our failure is a mix of general shear and local shear, and we estimate the bearing capacity by linearly interpolate in between the bearing capacity, that we obtain from general shear failure and local shear failure.

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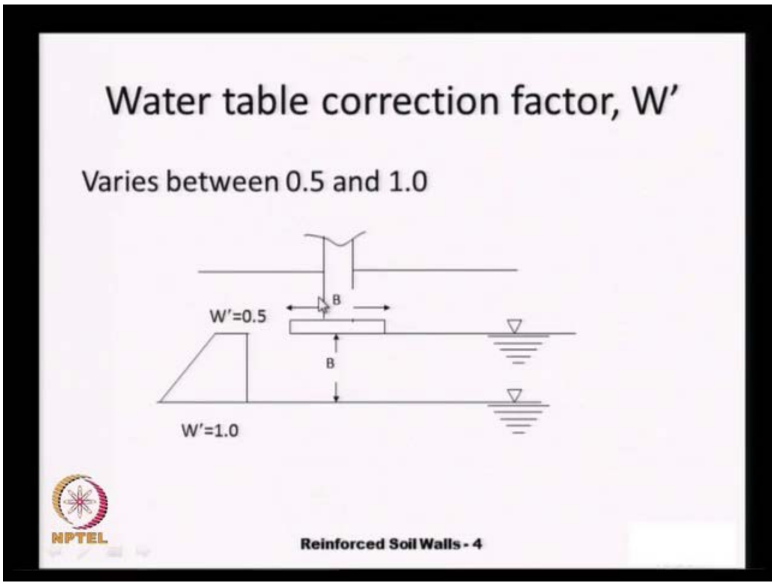

And the final correction that we apply is the w prime is actually, that is there only for the n gamma that is cause the gamma here is gamma bulk.

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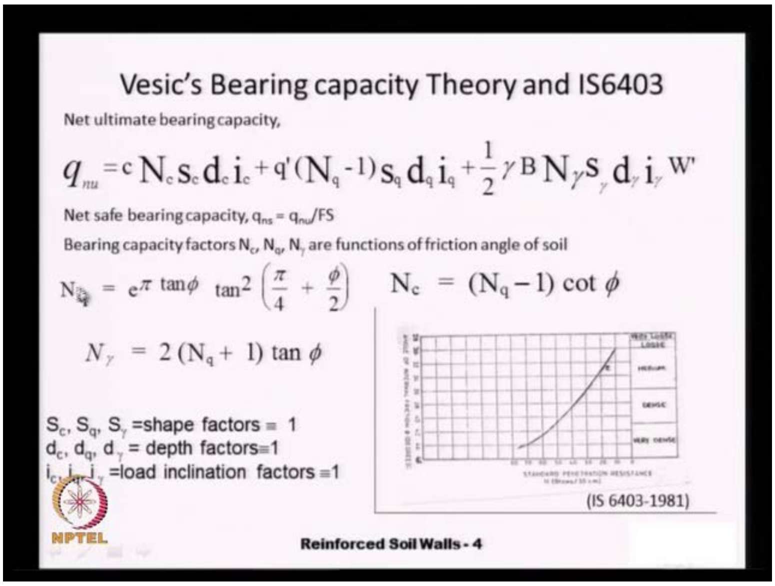

The bulk unit weight and because of that the w prime is applied to correct for the affect of water table, because if the water table is high we may have submansity unit weight and so t is applied like this.

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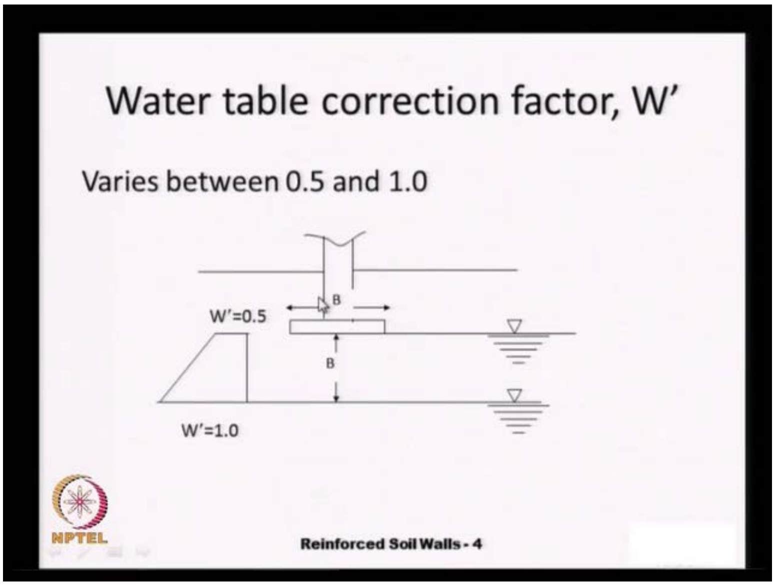

If the water table is at a depth of B or greater our water table correction factor is 1; that means, that we do not apply any correction, and if the water table is at the footing level or above the correction factor is half 0.5. This 0.5 is taken for the fact that the submerged the ration between the affective unit where that is the submerged unit weight and the bulk density the saturated unit weight is approximately half.

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The other aspect that we need to consider are these settlements and there could be different verities of settlements depending on the type of soil and the three major type of settlements that we have or the immediate settlements, the primary consolidation settlements and the secondary consolidation settlements. And this actually you the procedure for estimating these settlements is the same as that we follow for shallow foundations, and once again for the purpose of estimating the settlements we treat the reinforced block as the strip footing and apply all the all the formulas that we have.

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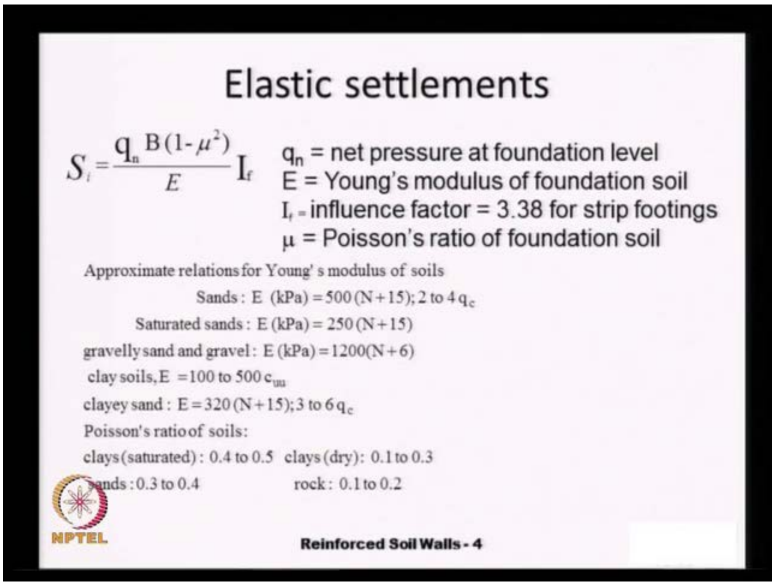

The of the first one is the immediate settlement or the elastic settlement, basically because we apply the elastic formula we call it also as an elastic settlements, it does not mean that the settlement is fully recovered if you apply if you remove the load. And the formula that we have that is given the I S 8009 is q n that is the net pressure B times 1 minus mu square by e times I f. Where, the B is the width of the footing, in this case that is equal to the length of the reinforced block l, and the I f is the influence factor the different quantities here the q n is the net bearing pressure the foundation soil.

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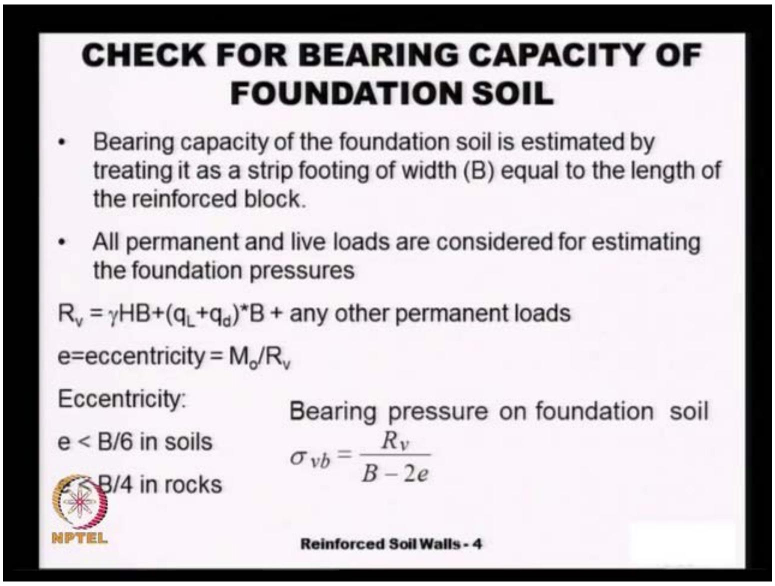

And this q n is noting, but this sigma v b that we calculate as the bearing pressure on the foundation soil.

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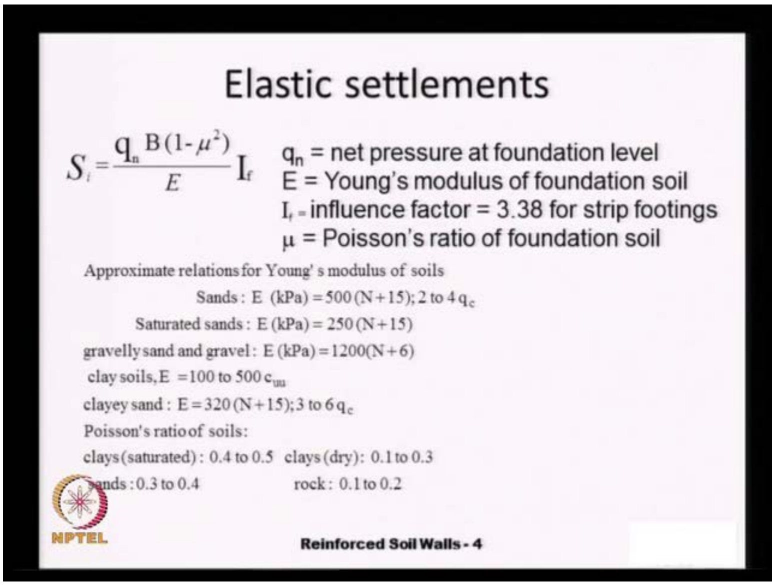

And the B is the width or the length of the reinforced the block and mu is the Poisson's ratio and e is the young's modulus, and I f is the influence factor and for strip footings it is 3.38 and the young's modulus e is related to different field the investigation values that we get. For example, B is related to the SPTN values like this for sands e is 500 times n plus 15 or for saturated sands the e is slightly less at 250 times n plus 15 and so on. And once again the Poisson's ratio they depend on the type of soil that we have either the clay soil, and if the soil is fully saturated the Poisson's ratio could be very near to 0.5 and if it is dry clay it could be very low of the order of 0.1.

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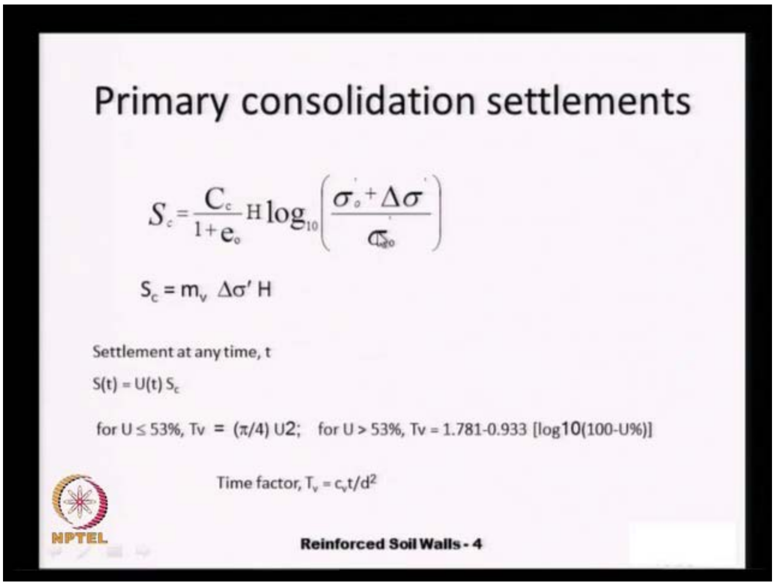

The primary consolidation settlements they are estimated either in terms of the compression index C c are the coefficient of volume compressibility m v depending on the data that we have. If we are within the normally consolidated rains we apply the first formula C c and if we are in the over consolidated rains we apply the m v, and the different factors here.

The e naught is the initial void ratio h is the thickness of the foundation layer, and this is not the height of the retaining wall, but it is the thickness of the of the foundation soil. And sigma naught prime is the existing effective over burden pressure in the foundation soil, and delta sigma prime is the increase in the over increase in the pressures because of our construction purpose.

Like for example, the sigma v b at different depths, that sigma v b is of the at the surface of the foundation soil and at deep depths that may reduce, and for the purpose of calculating the pressures in the foundation soil, we assume 2 vertical to one horizontal dispersion of the surface loads. And these settlements are the consolidation settlements and depending on the service life of the structure, we will have to apply some correction that is the degree of consolidation, and the degree of consolidation, we can get from our Terzaghi equations in terms of the time factor.

The time factor is c v t by d square that is where c v is the coefficient of the consolidation, d is the dryness path length, and t is the time and the procedure of these calculations is exactly the same as how we calculate in the settlements of shallow foundations.

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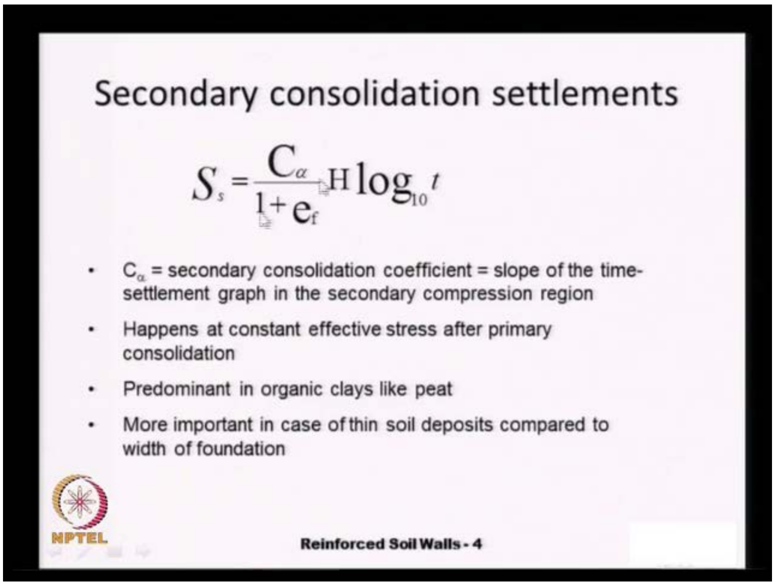

The other type of settlement that we have is the secondary consolidation settlements, that is the S is c alpha by 1 plus c f h times log t the secondary consolidation settlements are time dependent, they are not dependent on the pressure. Whereas, the primary consolidation settlements they depend on the pressure increments, see here we have the pressures whereas, here we do not have the pressure, but we have only the time.

The C alpha is the secondary consolidation coefficient that is slope of the time settlement graph of the consolidation data, beyond the primary consolidations. And then these secondary consol consolidation settlements, they happen it constant effective stresses after the primary consolidation settlements and this type of settlements that predominant in the case of organic place.

And if our foundation soil investigation says that we have organic place, then we will have to apply this correction you apply this settlements, otherwise we do not have to do that. And the secondary consolidation settlements they are very significant if your foundation soil is thin as compared to the width of the footing that we have.

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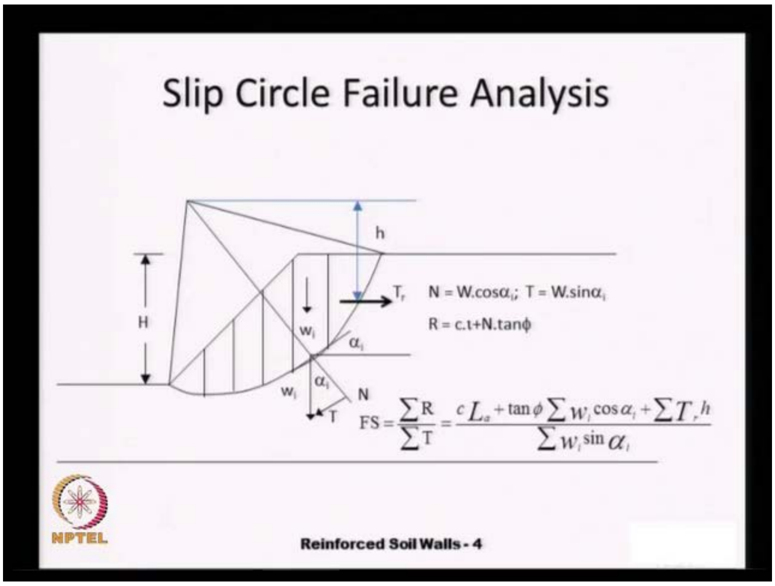

And the final check that we have after we satisfy the stability against the lateral sliding and against overturning and the bearing capacity or the bearing pressure that we apply is the slip circle failure are the overall the failure. Is actually typically the procedure that we apply is the just as how we apply the slip circle analysis for the slopes, and the only modification that we have is we account for the additional resistance force that we get form the reinforcement layers. And this we will discuss in more detail when we go the design of the retaining walls, but I will not delve too much in this lecture, but the procedure is like this.

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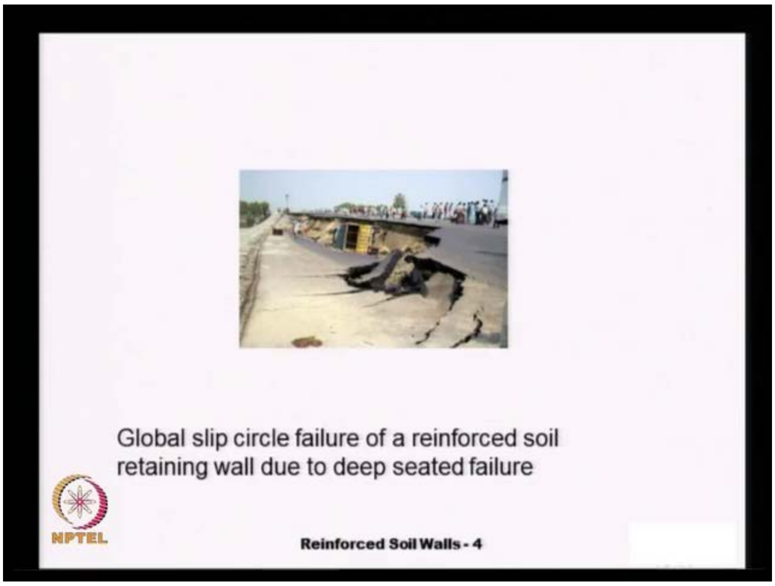

And what happens, if we do not consider the foundation soil, is actually here you see an example of what happens is actually it is about to 8 to 9 meters high approach road, that is supported by reinforce soil retaining wall system. That is with a foundation soil that is predominantly soft clay, and this particular retaining wall it is satisfies all the external stability requirements like the sliding overturning and the bearing pressure, but unfortunately in this case they did not do the deep seated failure analysis.

And this is what happened one fine day when the road is in service is actually we can have a disaster like this, and these can be avoided only by doing the last check that is the stability against the slip circle failure.

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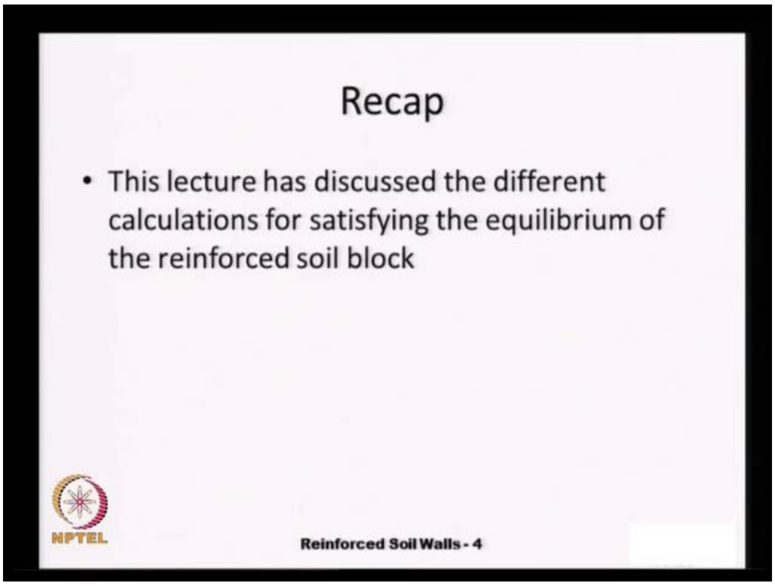

And just a recap we have discussed the different steps that are involved in calculating the length of the reinforced block to satisfy all the of the different external stability requirements, that is the sliding, overturning, bearing capacity failure, of the settlements and then the slip circle failure.

Thank you very much.