Geosynthetics and Reinforced Soil Structures
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Lecture - 18
Design Example of Reinforced Soil Retaining Walls – IV

So, very good morning students in the previous classes, we have seen the the calculation of the pullout, and rapture resistance of the reinforcement layers subjected to the different type of loads.

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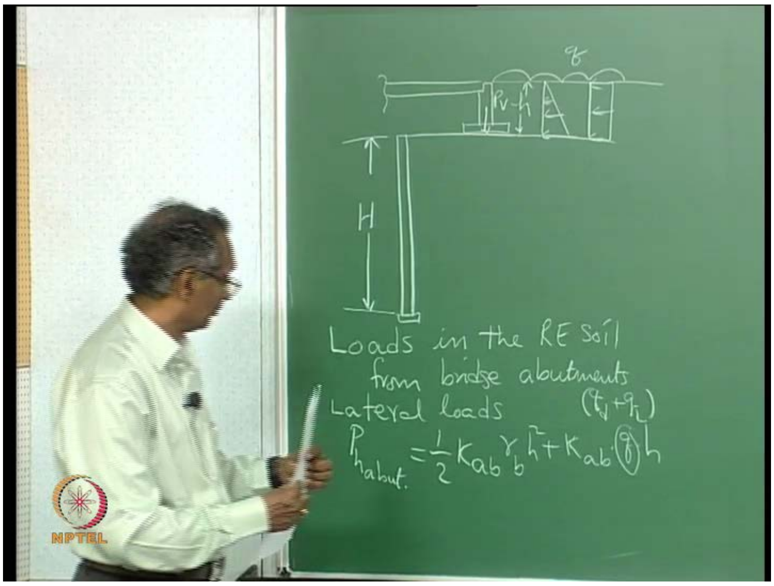

And now in this lecture let us look at the retaining walls directly supporting a a bridge abutment on top of the reinforced fill say as we have seen earlier a reinforced retaining wall is a something like this. And now let us assume that there is a a bridge abutment directly sitting on top of this something like this, and it is directly transferring a load of vertical load of P v. And then because of this a height of the soil h, there will be some additional lateral loads that are going to act on top of the on the retaining wall. And Let us say that there is some load of some uniform surcharge of q it could consist of dead load and alive load. And the way we treat dead load and live load we have already seen earlier that the all the loads are considered for calculating the force on the system, but when it comes to resistance, we neglect the contribution of the live loads and the same concept is also applicable here let us let us title it as. So, the loads let us first look at the lateral loads and let say that the height of the soil that is placed on top of the reinforced fill is small hits while the the height of the retaining wall itself is capital H.

So, our the we can say that the lateral load, because of the abutment is something like this one half K a b gamma b h square plus the one half K a b gamma b h square is, because of the self weight of the soil just directly behind the bridge abutment. And then the K a b times q h is the rectangular pressure distribution that is directly acting behind the bridge abutment because of the uniform Surcharger. And as I said before this q consists of the dead load plus live load, and we need to treat them appropriately as you require for the force cause in the failure we consider both dead load.

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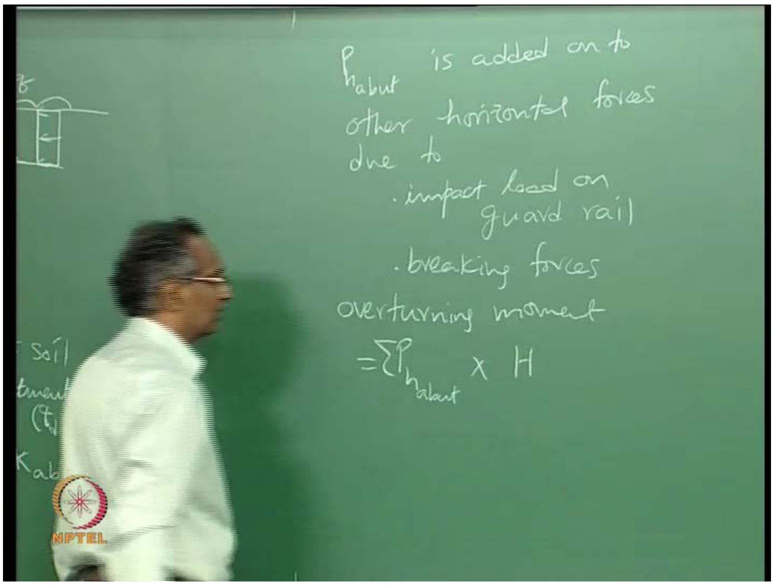

And live load and for resistance only the dead load is considered and this force this P h abutment is added on to. So, these horizontal forces that are acting on the bridge abutment we add them on to the other horizontal forces arising from the from the impact load on the guard rail. And then the breaking forces, because all of them are acting on the same horizontal direction we just combine them together and then the overturning moment moment is just simply the the sigma P h abutment multiplied by capital H because that is the lever arm for all these forces. And the in addition to this we have to also take care of the vertical load that is acting that is acting on directly on top of this this reinforce soil.

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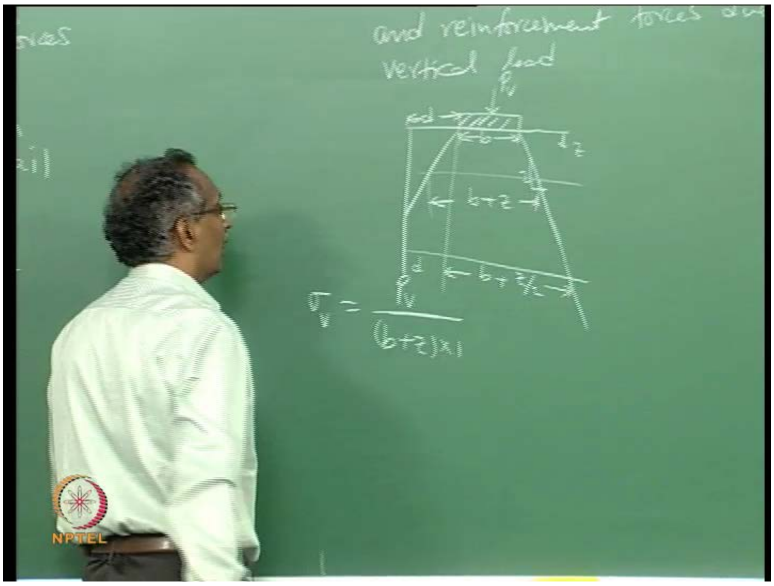

And now let us see how we can do it. See for this we assume a very simple load distribution we assume that the footing is of width b, and the load is directly acting through this footing. And we assume that this this vertical load is dispersed in to the soil at a an angle of two vertical one horizontal ratio and then at any level at any level Z we can calculate this as b plus Z. And then this sigma v is just simply the P v divided by b plus Z times one and and the front side the load dispersion will stop at a when the when this distance exceeds d because because of the abstraction from the from the wall facing.

So, beyond beyond this depth we calculate the this width as d plus b plus Z by two and it is a very simple calculation, and this calculation is very similar to what we do in a in the foundation engineering calculations to the calculate the pressures behind the below the shallow footings. And now let us give some numerical values and then do some calculations.

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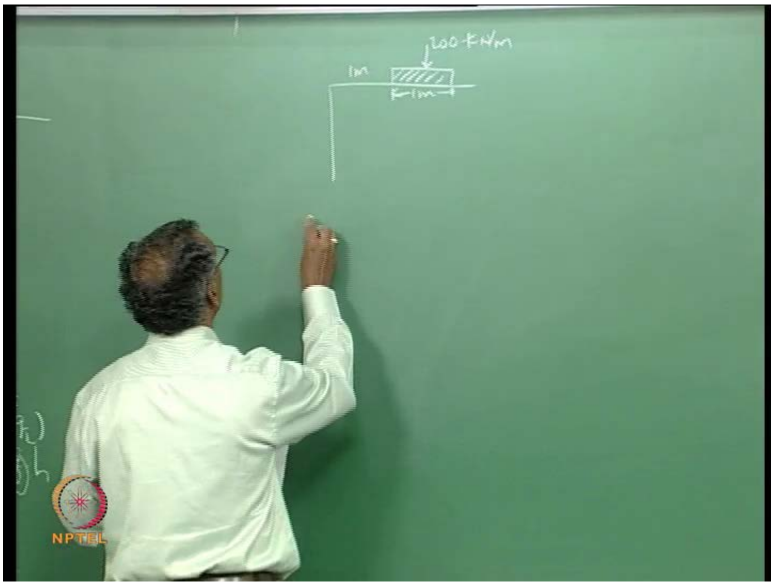

Let us assume that we have a footing f width 1 meter and with a frontal distance of 1 meter behind the retaining wall, and there is a load acting of 200 kilo newtons per meter and now we want to calculate the forces that are transferred into different layers. And we have already seen the layout of the different reinforcement layers there were totally 9 layers at different depths.

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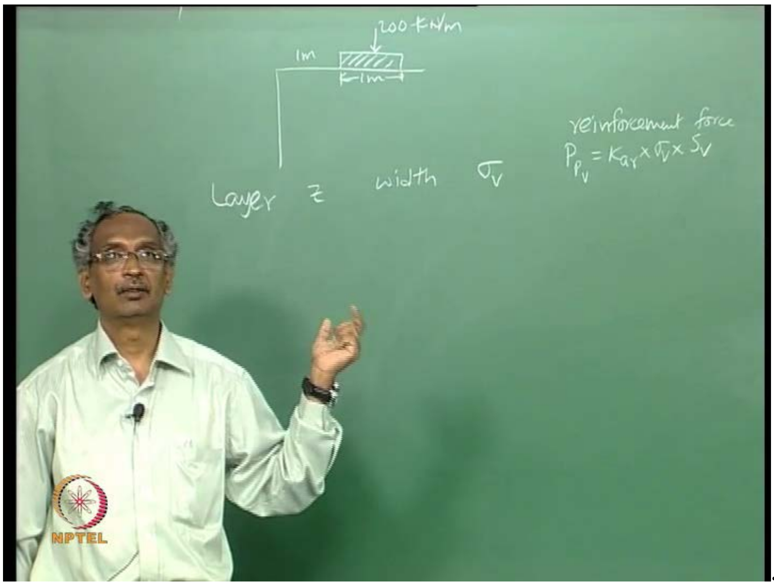

The load in the reinforcement layer reinforcement force is calculated as K a r that is the a lateral earth pressure coefficient within the reinforced fill. And for simple case we have seen that that is equal to the Rankine active earth pressure coefficient based on the friction angle of the reinforced fill multiplied by the sigma v that we are going to calculate.

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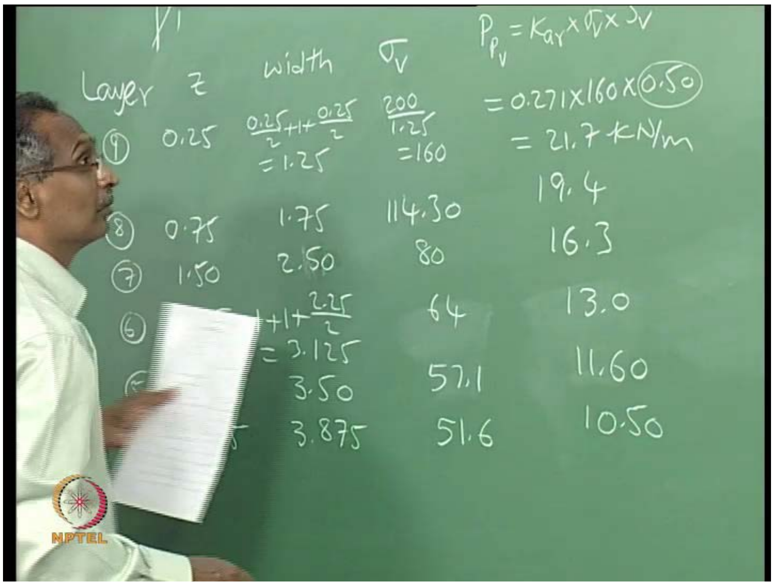

Now and S v is the the contributory area corresponding to each different layer, and now let us calculate layer wise the we have already seen that the layer number 9 is the top most one. And it is at depth of 0.25 meters and the width of the of the imaginary footing at a depth of 0.25 meters is 0.25 by 2 plus 1 plus 0.25 by 2, that is 1.25. And the sigma v is 200 divided by 1.25 that is 160 K p a, and we have already seen that the K a r is 1 minus sin 35 by 1 plus sin 35 that is 0.271. So, the load transferred into the top most reinforcement layer is 21.7 kilo Newtons per meter, because of the of the bridge abutment load vertical load on the bridge abutment, and this load we have to add on to the other forces that we have calculated earlier.

And this 0.5 is the contributory area corresponding to the top most reinforcement layer and because it is at the top the entire soil above this layer contributes to the force within this layer that is 0.25. And then the vertical spacing below the top layer is 0.5 meters. So, that 0.5 divided by 2 added on 2.25 becomes 0.5. And similarly the the eighth reinforcement layer is at depth of 0.75 and this width is 1.75, and this is 114.3 194, sorry 2.5 then the sixth layer is at a depth of 2.25 meters from the top. And when we dispersed the load on this side at a ratio of two vertical, one horizontal the the imaginary width at a depth of 2.25 meters is by 2 that is greater than 1.

So, we limit this width to 1 and 1 is our footing width and this on the other side we allow the footing the imaginary width to to extent 2.25 by 2. So, that is 3.125 and this is 57.1 sorry this is 13 sorry this should be that should be 64. So, these are all the additional forces that we have because of the vertical load that is directly acting on top of the the reinforced fill and we need to add these forces to the already calculated forces and and then check for the pullout stability. And then the and then the fact of safety against rapture and in the previous cases we have assumed that the reinforcement is is a is a spread type reinforcement. That is that has continuous surface, but we may have some other type of reinforcements, that is steel strips and steel strips within anchor embedded at the back and so on

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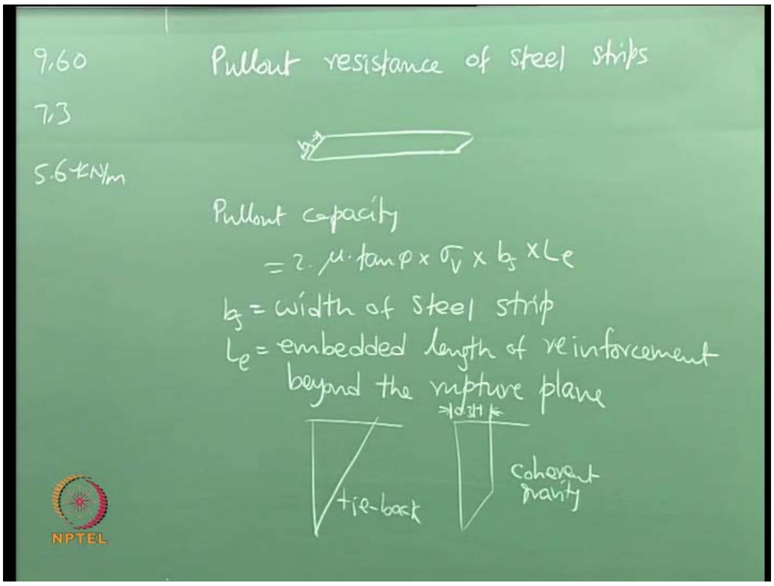

And let us briefly see how we can calculate the the pullout resistance of these steel strips, and these steel strips, they have a very small width of the order of 50 millimeters to 60 millimeters and let say that the width is b s. And our pullout capacity is that two times mu times tan phi times sigma v times b s times L e, where our b s is width of steel strip, and L e is the see these quantities. They are has defined earlier the b s is the width of the steel strip and L e is the embedded length of reinforcement beyond the rapture plane and just to recall we have two types of rapture planes the planar rapture plane. And then bilinear type rapture planes depending on the type of analysis in the case of tieback wedge method of analysis we have assumed that the rapture plane is planar surface at an angle of pi by 4 plus 5 by 2 that is Rankine act to surface. And then in the case of coherent gravity method of analysis the the filled the width is 0.3 h and in fact, the coherent gravity method of analysis applicable for rigid type reinforcement. And our steel strip is considered as rigid reinforcement, because it develops its its peak rapture capacity at a very small strain any reinforcement that develops the the peak strength at a strain less than 1 percent is treated as a as a rigid reinforcement, and is actually we all know that the steel has very high yield strength.

So, the calculated pullout capacity is going to be very, very small compared to the tensile strength of the reinforcement; that means, that we are burying a very expensive material,, but we are not able to utilize the full strength of the material and to overcome that we can add some anchor elements to this steel strip. So, that we can increase the pullout capacity of this this steel strips.

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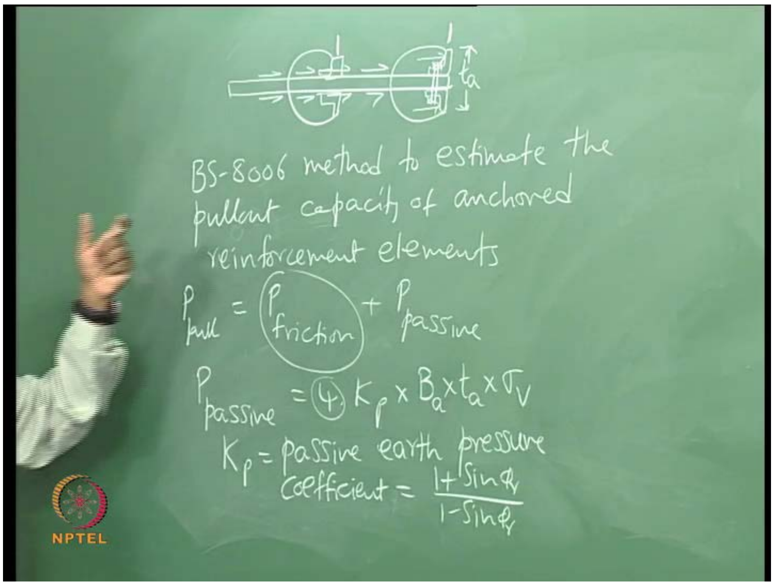

And how we do that is in cross section. So, the simplest way to add an anchories just take an l anchor, and then bolt them at the at the end up the the end of this steel strip both one at the top, and one at the bottom. And let say that the height of this anchor is t a and our now the pullout capacity is because of the because of the share resistance, that is developed on the length of the length of the surface. And then the passive resistance that is developed developed, because of the anchor the BS say among all the codes only the BS code BS 8006 code gives us a methodology to estimate the pullout capacity of the anchored elements reinforced anchored elements here. The P pull is because of the the frictional capacity plus P passive passive capacity, because of the passive pressures that are directly acting against these anchor elements.

And that the P friction capacity is the same as, what we have seen earlier; that is 2 times mu times tan phi times the surface area multiplied by sigma v. And the passive capacity is it is given as 4 times K p times B anchor times t anchor times sigma v, where our K p is the passive pressure coefficient. So, this K p is the passive earth pressure coefficient that is just simply 1 plus sin phi r by 1 minus sin phi r and this factor 4 is actually say very highly conservative value usually it is about at least 8 to 10 times more than more than this this this value. We will see some experimental data later on comparing the the predictions from the BS formula to the actual pullout forces that that we have done on the anchored elements and here the B a is the width of the anchor, and t a is the height of the anchor.

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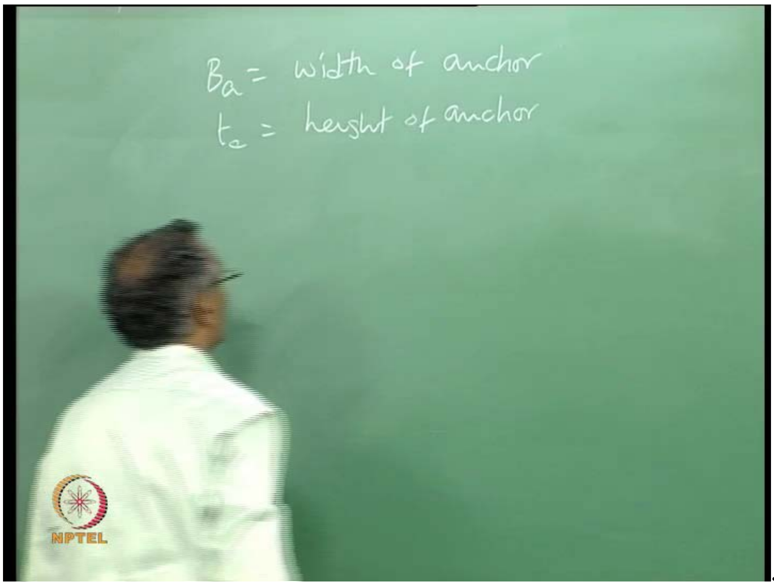

So, the B a times t a is basically the exposed surface on which the passive pressure is acting, and depending on the and the requirement, we may have 1 single passive element or we may have multiple passive elements like this. Because some companies they utilize multi anchored reinforcement elements, whereas some companies they they have only one single anchor at the end, and depending on the and the spacing that we have between the anchors this the the passive capacity should be estimated the appropriately is they are very closed together. We cannot just simply increase the the passive capacity by the number of anchors that we have we may have to reduce, because of the because of the interaction that is taking place between the different anchors. And usually these codes they do not give formulas for more complicated systems except for suggesting that the for any specific or for any special case, we need to confirm them confirm the capacity by doing a some full scale pullout test. And it is best to estimate the pullout capacity of these multi anchored elements by doing some test to establish what is the minimum spacing that we require. So, that there is not too much of an interaction between the between the these passive elements that is see let us say that.

So, much of soil is affected by by the passive pressure in front if these anchors and if these anchor elements are too closed together the pressure bulks that are formed inter in front of the anchors they interact, and because of that the efficiency reduces. So, that we can only determine by doing some test because the size of this pressure bulk also depends on on the soil properties that we have and then the amount of compaction that we have given.

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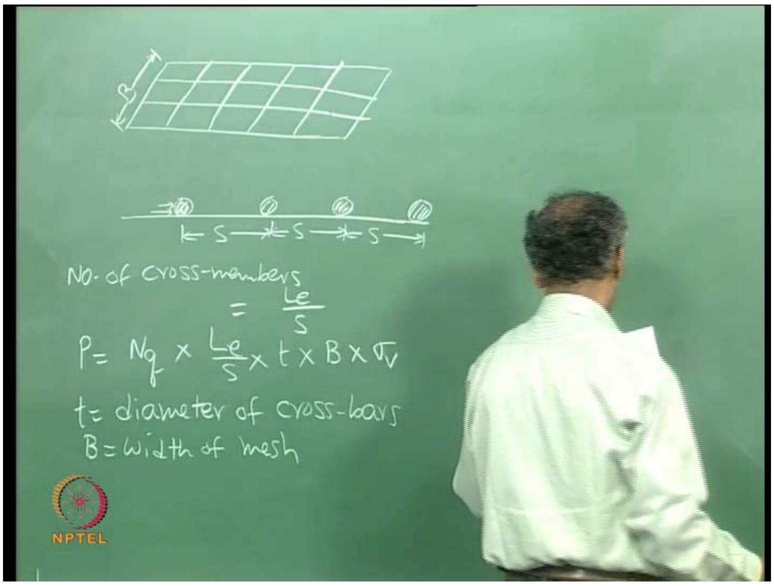

So, on the other type of element, other type of reinforcement that we need to specially consider is the is the welded wire machines see in section, they may look something like this, and their pullout capacity is not only because the friction that is developed, but also because of the passive pressure that is acting against against. These the projected surfaces and let us say that these this spacing the horizontal spacing between these cross wires is s. And the number of see if L e is our embedded length of the reinforcement L e divided by the center to center spacing between these cross members is the number of number of cross members that we have. And then our P is N q N q is the bearing capacity factor that is that is developed for for the horizontal pull on this on this welded wire machines, where a t is the diameter of this cross members and B is the B is the width of this welded wire mesh.

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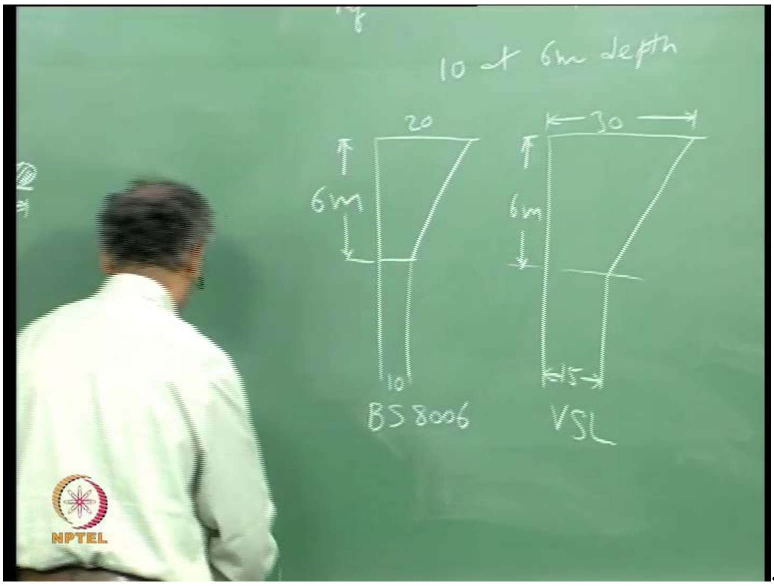

And of course, sigma v is the pressure and N q it is 20 the N q is similar to our bearing capacity factor, and if the BS code the N q is recommended as a 20 at the top the soil surface. And then value of 10 at a depth of 6 meters, but some companies like VSL they take it as much higher they take this value as 30 at the top surface, and reduce into 15 at a depth of 6 meters. Because the BS code is highly conservative, as you can see because the experimental observation is the N q's very very high of the order of 40 to 50 at the top surface and 1 at depth of 6 meters it is much more than 30. So, the VSL VSL corporation uses this N q factor of 30 of the surface and 15 at the at depth of 6 meters and it is a good practice to verify this values, because sometimes our soil may have very high fines content. And if we have very high fines content the passive pressure coefficient could be smaller than what we expect, and it is a it is a good practice to perform some full scale pullout test and confirm that whatever pullout capacity factors that we are using their appropriate, what happens? If we do not have any mechanism of experimentally verifying the pullout factors, and the some codes they give some empirical formulas for for estimating the pullout factors based on the soil properties.

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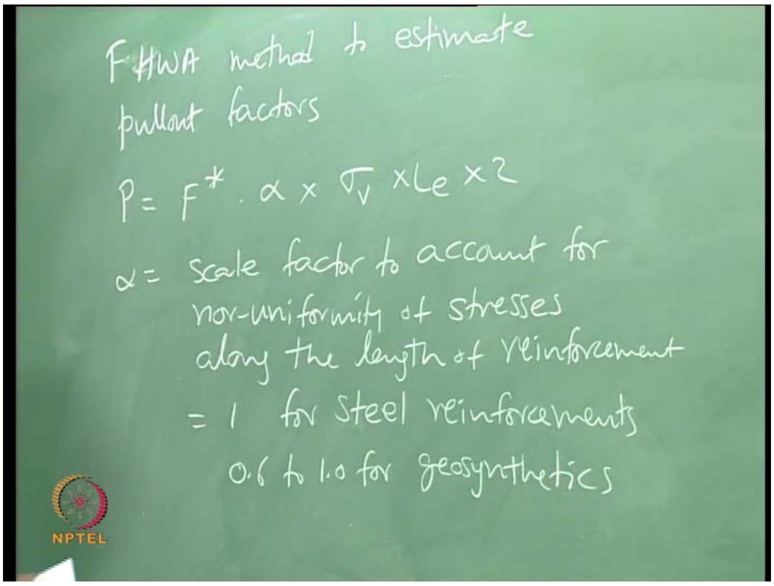

And one such formula is given by FHWA see in the FHWA method, they suggest that the pullout capacity be estimated like this F star times alpha times sigma v times L e times 2; that 2 is for two surfaces - one at the top, and one at the bottom. And our alpha is the is the scale factor to account for for non-uniformity of stresses along the length of reinforcement. And usually it is taken as one for steel steel strips or steel reinforcements and about 0.6 to 1 for geosynthetics, if we have very stiff geosynthetic or a polymeric reinforcement we could take an alpha of 1, and if we have a very highly flexible reinforcement that has a very low tensile strength.

We could have a value of even lesser 0.6, because of the if the reinforcement is highly extensible, then it is shown that along the length of reinforcement at the vertical pressures are not constant. They are also affected by the reinforcement strains and to account for that factor we use an alpha,, but in the BS code an alpha of 1 is used. In fact, they do not have that; that means, that they assume a uniform vertical pressure along the length of reinforcement and our sigma v is the vertical pressure L e is the embedded length.

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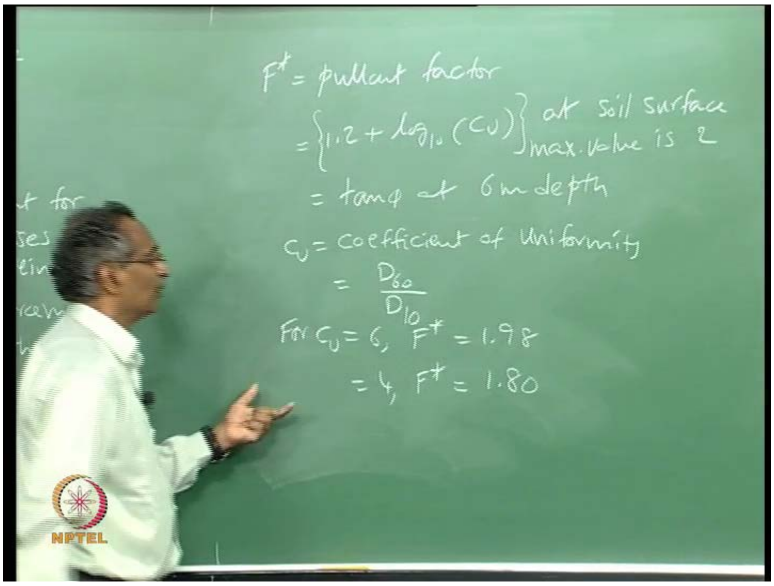

And F star is the pullout factor that is taken as 1.2 plus log of C u at the top surface at the soil surface and it is taken as tan phi at 6 meters depth, where our C u is the coefficient of uniformity. That is D 60 by D 10 say for for C u of 6 the F star is approximately 1.98 that is very close to, and this in fact this value maximum value recommended is 2, because for a C u of 6 the F star comes to very closed to 2, that is 1.98. And the for a C u of 4 F star is 1.8. So, actually from this you can see that the at very low normal pressures the pullout could be very very high, because of the effect of dilation. So, these are some of the additional methods to estimate the at the pullout capacity of the reinforcement layers apart from all these internal stability calculation that people form. We have to also check for one extreme case especially in highly loaded bridge abutments for the safety of wedge.

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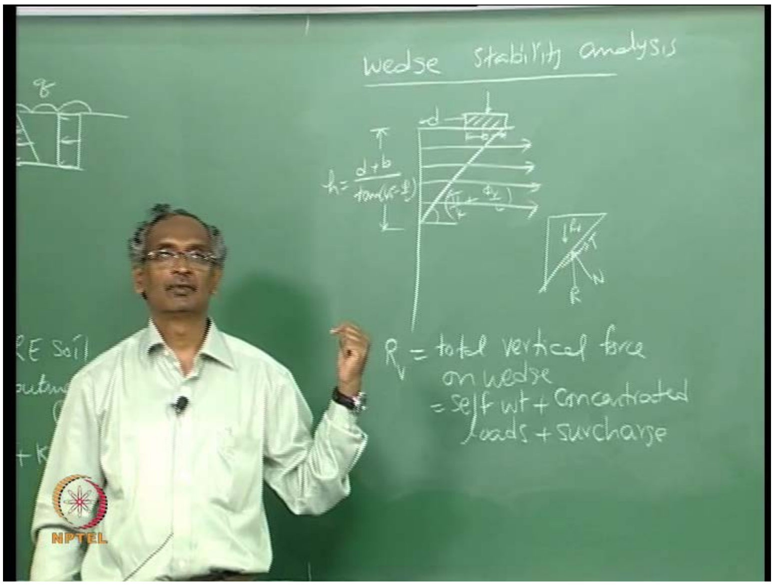

And that we call as wedge stability check or wedge stability analysis, we assume that the most critical Rankine failure plane happens just behind the behind the bridge abutment, and this we draw it an angle of pi by 4 plus phi r by 2. And and then we have to make sure that the number of reinforcement layers that are provided within with in this wedge they are sufficient to to stabilize this most critical wedge, see this the height of this critical wedge is d plus b by tan 45 minus phi by 2. And our if our R v is the the total vertical force is on wedge due to self weight plus concentrated loads plus surcharge of its.

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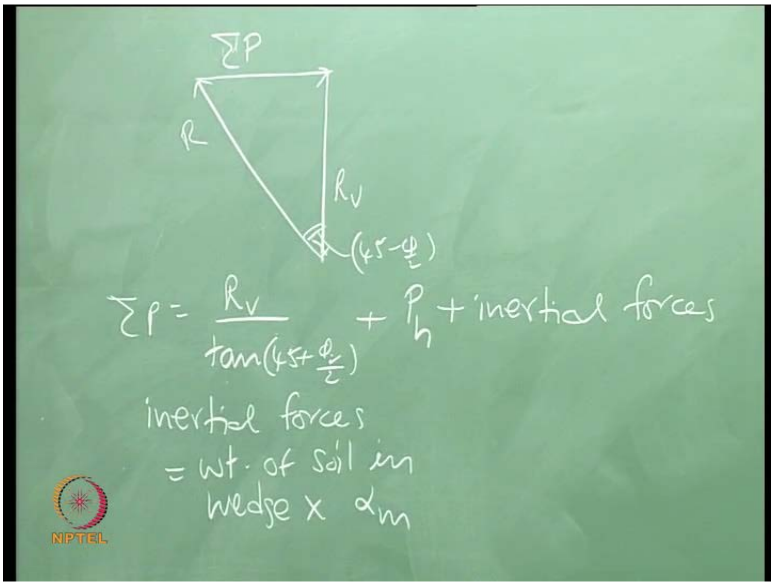

We can draw a force force polygon, and say see by by considering a the force equilibrium like this say we just as how we do in the lateral earth pressures by the columbus method. We can draw the force polygon like this and then calculate the sigma P that is required the and the reinforcement layers as R v divided by tan 45 plus phi r by 2 and this should be added with other horizontal forces, that are acting on the bridge the the reinforced fill that is the P h, that is these are all the horizontal force that are acting plus the inertial force in. And the inertial forces or the just weight of the soil in wedge multiplied by multiplied by this alpha m alpha m is the is the calculated inertial coefficient within the reinforced fill that we have estimated earlier as 1.45 minus alpha times alpha that is the alpha m and for this particular case. Let us look let us work out some members.

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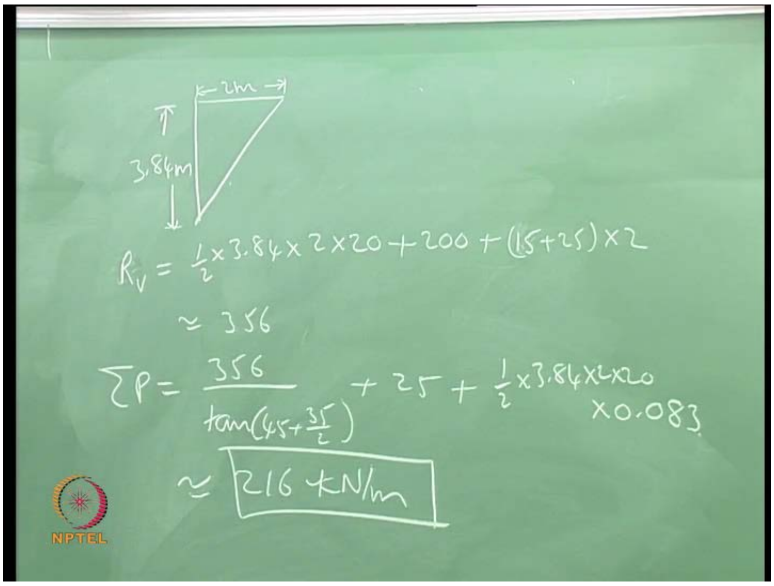

The total vertical force R v is the weight of soil within this wedge that is one half 3.84 times 2 that is the top width multiplied by 20 is the unit weight of the soil plus 200. That is the vertical constituted force plus the the total surcharges that is the dead load and then the live loads surcharges multiplied by 2. So, our this entire force is approximately 356. So, our this sigma P is is 356 divided by tan 45 plus 35 by 2 plus. We have a horizontal force acting of 25 kilo newtons per meter, and the inertial force is one half 3.84 times 2 times 20 multiplied by 0.083, and this comes to approximately 216 kilo newtons per meter. So, this force we need to compare against the number of reinforcement layers that we have within the within the top 3.84 meters.

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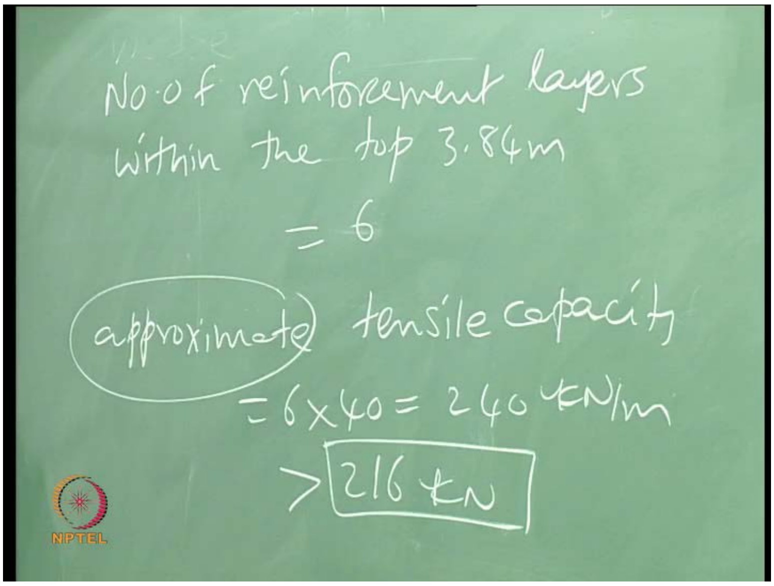

And that we will see the number of layers see you can just simply count the number of layers within the top 3.84 meters of wedge that is 6. And the approximate approximate tensile capacity is 6 times 40; that is the 240 kilo newtons per meter I have put approximate, because I do not remember. Whether the top most layer has a pullout capacity equal to the tensile strength see actual the the total capacity given by the reinforcement is the sum total of either the pullout capacity or the rapture capacity which one whichever one is lower, and is is approximately 240, which is greater than 216 kilo newtons.

So, the the wedge is stable as for as the all the lateral forces and then the inertial forces and then the the bridge abutment loads are concerned. So, this is how we check for the total stability of the of the retaining wall and in addition to all these calculations we also have to make sure that the if the the front facing is made of modular blocks or or individual facing panels the connection strength should be adequates. So, there is no rapture at the at the connection point. So, let us know a look at a very simple spread sheet program because all this calculations are very cumbersome, and we need to go through some repeated calculations let us briefly look at an excel spread sheet program.

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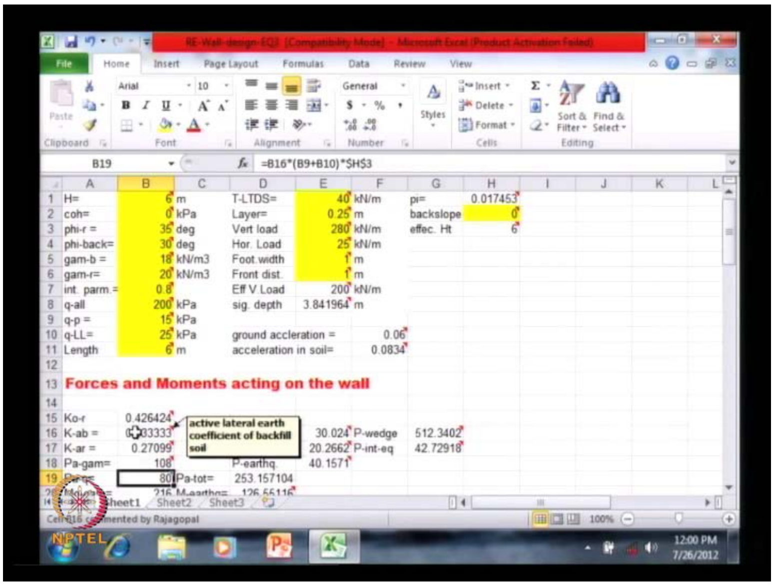

So, you will all get a copy of this excel spread sheet programs. So, that you can do this calculations yourself and let us this program requires some minimal input data that is the height of the wall that is 6 meters. In this case the cohesion of the the backfill material and then the friction angle of the reinforced fill then the friction angle of backfill and then the unit weight of the backfill, and reinforced fills. And then the interaction parameter that is the the mu in our case the pullout capacity is written as nu times tan phi and the mu of both 0.88 is is acceptable. And then our other properties that you require the long term allowable design strength of the reinforcement that is 40 kilo newtons per meter and these layers are provided at 0.25 meters vertical spacing. And then the back slope angle is 0, because we have considered a horizontal backfill and our allowable bearing pressure, and the foundation soil I have just taken as 400.

I think we have used a value of 200 in the in our designs and now let us see how we can and of course, our the on the permanent surcharge uniform surcharge is taken as 15 the live load surcharge is 25 and and now need to check for the the external stability.

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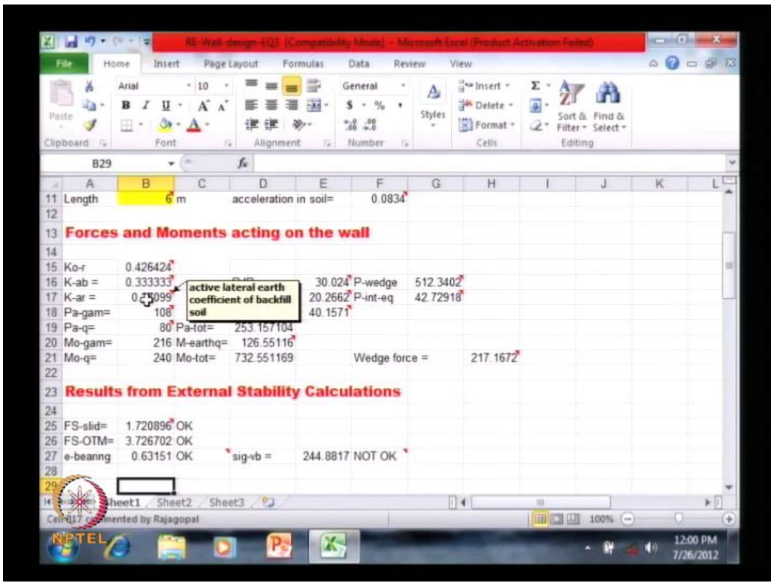

And so on and our these are the forces at the moments acting on the wall the the K naught is 1 minus sin phi. And then the K a b is 1 minus sin 5 by 1 plus sin phi that is one third and then the K a r; that is active lateral earth pressure coefficient of the reinforced fill that is 0.271. And these are all the the values that we have calculated earlier and then the other input that we need to give is the ground acceleration that is given as 0.06.

Then the acceleration within the reinforced fill is 1.45 minus alpha multiplied by a alpha that comes to 0.083, and these are all the the factors that we have and the fact of safety against sliding is 1.72. And then the overturning moment overturning fact of safety, and then the sigma v b these are all the different values that we have calculated. In fact, this 244 is by considering the the the abutment load, and for this particular case see for the when we did the hand calculations, we did not consider a the abutmental loads let us let me just give that vertical load as 0. And we have seen that the the pressure is 214 when our reinforcement length is 6 meters, and if we increase the reinforcement length let us say to 7 meters our sigma v b comes out as 196 which is less than the allowable bearing pressures. So,. So, it is safe well let me just go back and make it 6 meters.

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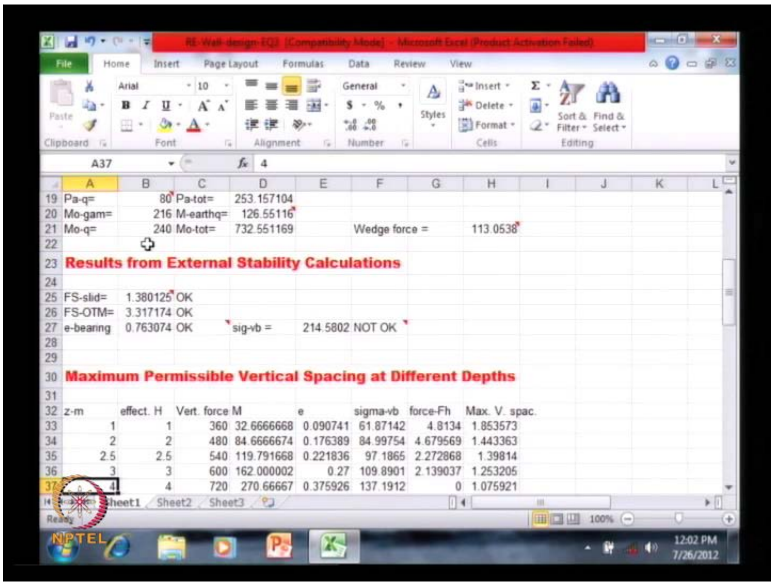

We need to calculate the length of reinforcement. So, that our fact of safety against lateral sliding overturning and then the bearing pressures are satisfied, and the results that we see here the indicate this. And as we have seen the fact of safety that we require and and a seismic load is only 75 percent of the static fact of safety. So, in this case the fact of safety against sliding of 1.5 is required for static loading 1.5 times 0.75 that is only 1.125 for lateral sliding. So, it is 1.38 which is more than 1.125 and these are all the the different vertical spacing's that are required at different depths I will not go through them, but since we have done the detailed calculations, you can check them yourself.

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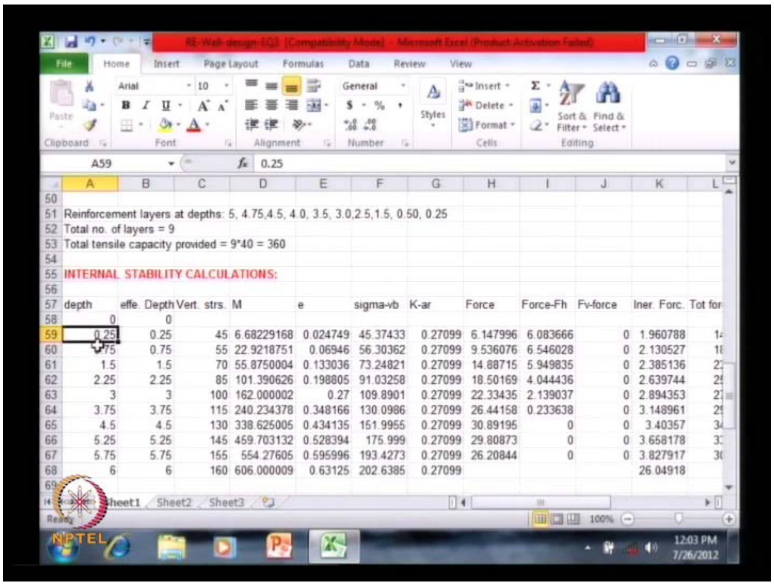

And these are all the different layers that we have provided totally 9 reinforcement layers the top most one is at 0.25, next one is at 0.75 and so on up to 5.75 meters and the vertical stress is gamma Z plus q. And then the overturning moment because of the because of the backfill earth pressure, and then the extrinsity sigma v b and the K a r is if your wall height is less than 6 meters. And if you have flexible type reinforcement it is just simply 1 minus sin phi r by 1 plus sin phi r that is 0.271. And then the force because of the self weight and the force because of the lateral force of 25 kilo newtons and then the inertial force that we have already calculated then this this F v force is because of the bridge abutment forces let me just turn it on. So, this is OK that is effective vertical load of 200 kilo newtons on the bridge abutment. So, these are the the forces in the reinforcement layers because of the abutment load the top most layer has 21.6 and so on.

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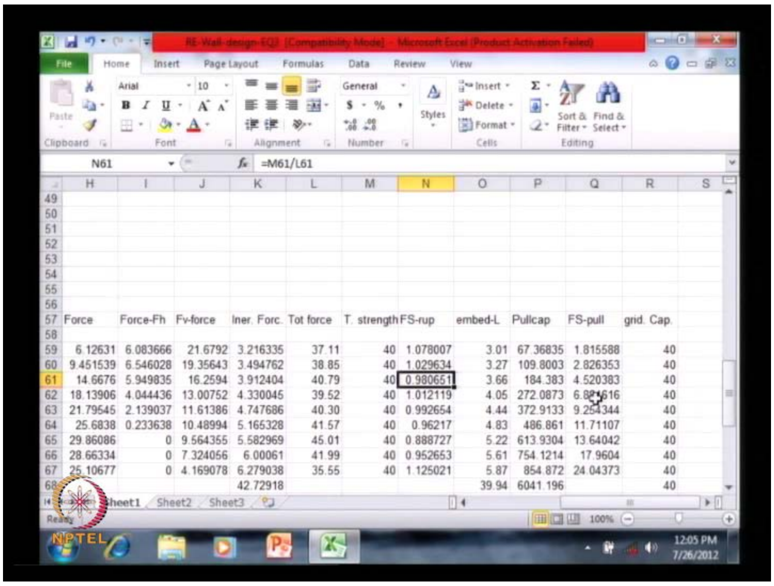

Row	Force	Force-Fh	Fv-force	Iner. Forc.	Tot force	T. strength	FS-rup	embed-L	Pullcap	FS-pull	grid. Cap.
59	6.12631	6.083666	21.6792	3.216335	37.11	40	1.078007	3.01	67.36835	1.815588	40
60	9.451539	6.546028	19.35643	3.494762	38.85	40	1.029634	3.27	109.8003	2.826353	40
61	14.6676	5.949835	16.2594	3.912404	40.79	40	0.980651	3.66	184.383	4.520383	40
62	18.13906	4.044436	13.00752	4.330045	39.52	40	1.012119	4.05	272.0873	[illegible]	40
63	21.79545	2.139037	11.61386	4.747686	40.30	40	0.992654	4.44	372.9133	9.254344	40
64	25.6838	0.233638	10.48994	5.165328	41.57	40	0.96217	4.83	486.861	11.71107	40
65	29.86086	0	9.564355	5.582969	45.01	40	0.888727	5.22	613.9304	13.64042	40
66	28.66334	0	7.324056	6.00061	41.99	40	0.952653	5.61	754.1214	17.9604	40
67	25.10677	0	4.169078	6.279038	35.55	40	1.125021	5.87	854.872	24.04373	40
68				42.72918				39.94	6041.196		40

Then the the factors of safety against the different pullout and then the the rapture the top most layer is the most susceptible one for pullout and it has the lowest fact of safety. So, 1.07 and the third layer is. In fact, it has fact of safety less than one that is because as we go down we have seen that the 0.75 meter spacing is starting from layer number 3 and the in this particular case this is the most critical one. And this means that we need to increase the increase the in the reinforcement strength or increase the length, and the sorry, in this particular case this is the fact of safety against rapture. And because the because the reinforcement force is more than the tensile strength we need to change the

the reinforcement type instead of having long term allowable design strength of 40 we may need to go and for reinforcement that has a highest strength.

Let us say of 50 kilo newtons per meter and so on. Then the factors of safety against pullout there calculated like this then the the grid capacity is the minimum of either the rapture strength or the pullout capacity, and in all these cases these are equal to the tensile strength; that means, that all the pullout capacities for different layers are more than the tensile strength of reinforcements. So, this is and then finally, we need to check for wedge stability, and the wedge force a by hand calculations we have got about 216,, but if you do it bit more regressly by not rounding of the numbers it comes to 217 kilo newtons per meter. And we have already seen that we have 6 layers of reinforcement that are acting across this active wedge that is drawn through the back of the back of the bridge abutment and so we have adequate tensile strength from the reinforcement layers. So, this particular design is safe except for the reinforcement capacity is slightly less than the the capacity that we calculate for this particular third layer, the we get a fact of safety of 0.98, and normally 0.98 is OK, we ignore it. We just because we have already reduce the reinforcement strength by different parameters like the instillation diametric the pre-prediction factor environmental degradation factors and so on.

We just allow it to go like we do not need to change the the reinforcement type,, but if this fact of safety against rapture at the pullout is much lower than one then we need to change OK. So, this program can be used for a doing some simple calculations for retaining walls that are subjected to self weight loads and then the bridge abutment loads and see if you have backfill slope you can give that. And it will the program will automatically do the calculations and then you can consider any height of wall let us say let us change this height of the wall to ten meters and will see that.

So, if you do that at the top the our K a is equal to 1 minus sin phi that is that is 0.42 this case the K factor goes on reducing with depth and that depth of a 6 meters the K factor is very closed to 0.27 is actually it is not come to that level. So, we see this the that the K is at depth of 5.756 0.287 whereas, a depth of 0.25 meters, this K a r is equal to very near to 1 minus sin phi. So, this is how we do these calculations, and I will give you some homework problems. So, that you can practice your designs both by hand and then you can check your design calculations using this excel spread sheet program.

Thank you very much.