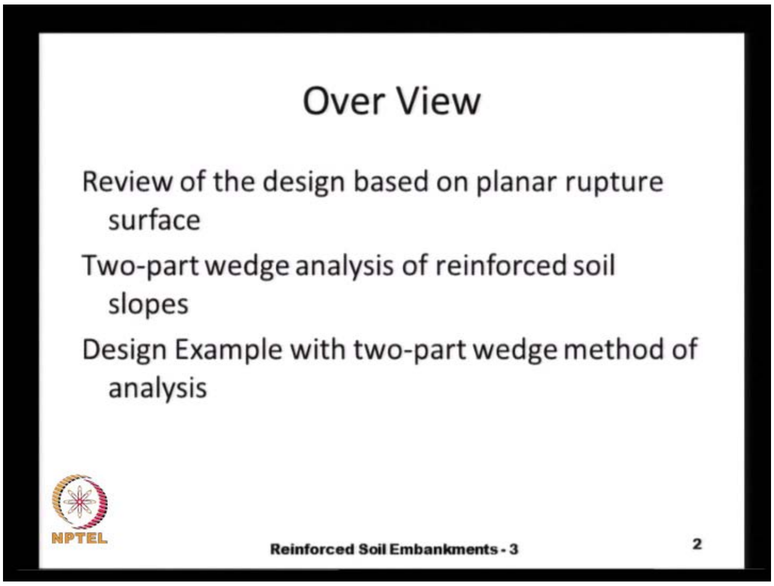
Geosynthetics and Reinforced Soil Structures
Prof. K. Rajagopal
Department of Civil Engineering
Indian Institute of Technology, Madras

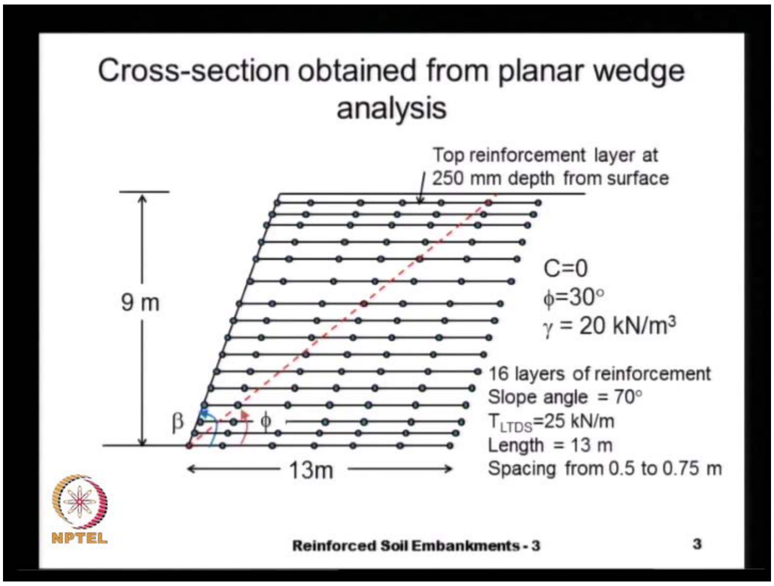
Lecture - 23
Two-Part Wedge Analysis of Reinforced Soil Embankments

(Refer Slide Time: 00:12)



Very good morning students, and a brief over view for today's lecture is we will review a the design that we have obtained based on the planar rapture surface. And then we look at at the two-part wedge method of analysis, and then we will also see the numerical design example based on the two-part wedge design.

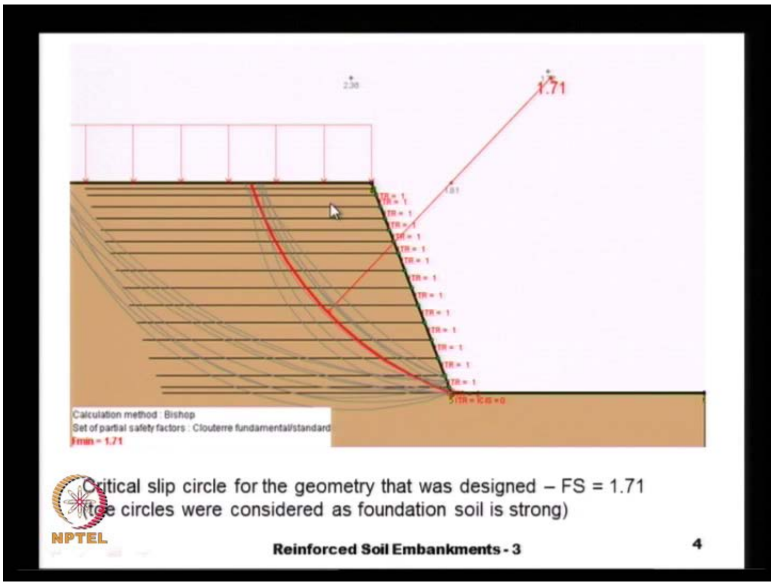
(Refer Slide Time: 00:32)



And this is the cross section that we have obtain based on the on the planar rapture surface, we have considered an embankment of height 9 meters having a slope of 70 degrees and the soil properties are C equal to 0, friction angle of 30 degrees. And unit weight of 20 kilo newtons per cubic meter then on the surface we had a uniform surcharge of 20 kilo pascals and the reinforcement that was considered had a long term allowable design strength of a 29 25 kilo newtons per meter. And our compaction layers were in multiples of 250 millimeters and the length of reinforcement we found that we require at least 13 meters.

Basically based on the on the minimum length that we require at the top layer and then to prevent any any base base failure. And the vertical spacing varied anyway from 500 millimeters to 750 millimeters; that is the bases that reinforcement force does not exceed the tensile strength of the of the material that is 25 kilo newtons per meter length, and the top we have provided one extra layer of reinforcement a 250 m m depth. Because that is the codal requirement, we should provide one reinforcement layer as closed to the top surface is possible to to take care of the surface stresses.

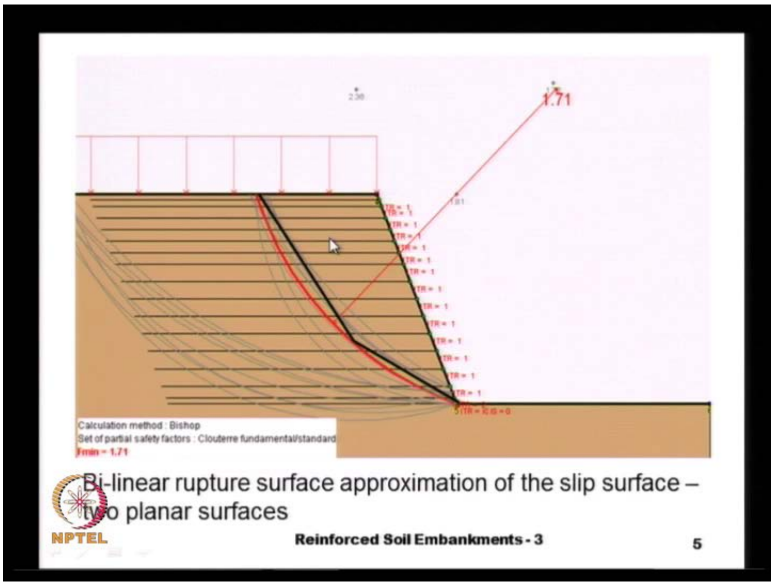
(Refer Slide Time: 02:27)



And the the slip circle analysis for the previous design is like this, this particular analysis was done by using the program tolerane. The height of the slope is 9 meters and this slope angle is 70 degrees and then a uniform surcharge of 20 k P a was applied on the on the on the crest. And because our foundation soil is a strong soil we will not have any deep seated failure circles and all the circles that will considered or to a circles and very large number of slip circles are analyzed. And the least factor of safety that is obtained is 1.71 that is for this circle that is shown in red passing through the toe and then intersecting this slope at some distance. And we see that all the reinforcement layers are providing resistance I guess the except the bottom most one, because the bottom most one it is not considered as affective. Because the slip circle is passing through the edge of this this particular layer and other than this layer all the other layers they do provide then overturning resistance and.

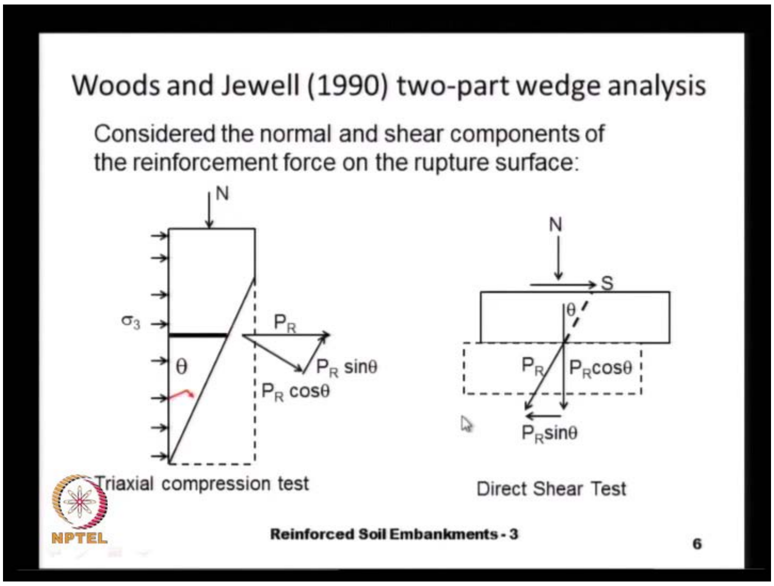
The factor of safety that is 1.7 which is weigh to high, because our the factor of safety that we normally provide for for road embankments and other embankments is about 1.4. So, we need to come out with better methods of design, because the assuming a planar wedge surface is is an extreme limit, because this particular angle that was considered is equal to the friction angle that is basically we have considered the angle of repose for the rapture surface which is extremely conservative. And so we we can think of a better slip surfaces like we can see here that slip surface, that is that has the least factor of safety that is 1.71 is not passing through the back of the the slope, but it is passing somewhere in between…

(Refer Slide Time: 05:09)



So, we can actually approximate this slip circle by using a bilinear wedge or or a two-part wedge method of analysis, and we also realize that most of these embankments or steep slopes. They are constructed using very good quality granular materials in which case the rapture surface is they are planar as assumed by both rankine and coulomb and rather than a circular slip surface. So, it is most appropriate. If we can analyze the safety of these embankments based on planar rapture surfaces.

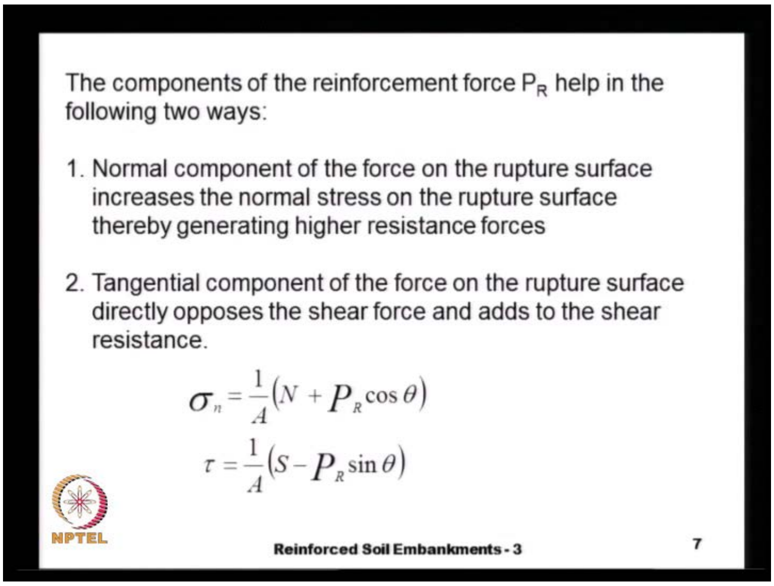
(Refer Slide Time: 05:50)



So, that is the subject of ah today’s presentation and woods and jewell in 1990 they published a paper based on the two-part wedge analysis of the reinforced soil slopes and the main difference between their analysis, and all the other previous analysis are that they considered the normal and shear components of the reinforcement force on the rupture surface. So, they considered that reinforcement does not just simply contribute the tensile force for resisting the for resisting the upsetting forces, but it also provides additional effects of normal and the shear shear components. And when we look at let say the triaxial compression test like this. So, we have a a samples cylindrical sample and with some reinforcement layers like this and as we are compressing the sample reinforcement does develop some force. Let us say P R and on this rupture surface we will have two components P R times cosine theta; that is the normal component and then P R sin theta is is the tangential component ,that is opposing the the shear forces directly and another situation in the case of direct shear test can be considered like this.

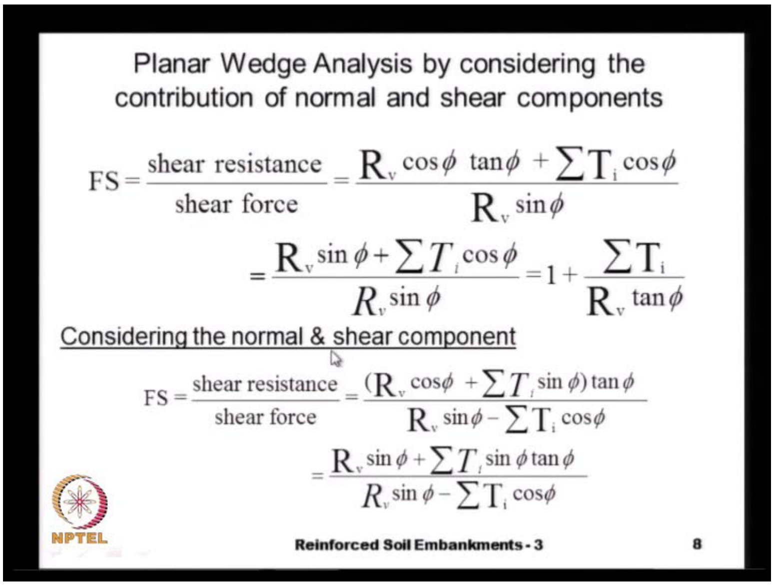
In fact, this was the P hd work of professor Jewell, he did lot of works on on the on the shear strength of soils in the direct shear box placing inclined reinforcement layers like this, and as the as the shear is happening the reinforcement develop some force and once again. We can we can take like components of this reinforcement force as normal component then a tangential component to this rupture surface the P R cosine theta is directly adding on to the on to the normal force; that is acting on the on the rupture surface. And then the P R sin theta is opposing the the shear shear deformation.

(Refer Slide Time: 08:18)



Woods and Jewell they considered that the components of the reinforcement force helps in the two ways; the normal component of the force on the rupture surface increases the normal stress on the rupture surface thereby generating higher a shear resistance ,because we know that the toe is C plus sigma N tan phi. If you are able to increase the sigma N we can mobilize higher and resistance forces, and then the tangential component of the force on the rupture surface directly opposes the shear force and adds to the shear resistance. And this is how with it goes the sigma N that is the normal stress on the on the rupture surface is 1 divided by A, where a is the is the shear area that is N plus P R cosine theta where N is the applied force. And P R cosine theta is the is the normal component of the reinforcement force on the on the rupture surface. And then the toe is 1 by A times s s is the shear force; that is the applied shear , because of the moment of direct shear boxes minus P R co P R sin theta, where P R sin theta is the tangential component along the rupture surface that is directly opposing the the shear force.

(Refer Slide Time: 09:47)



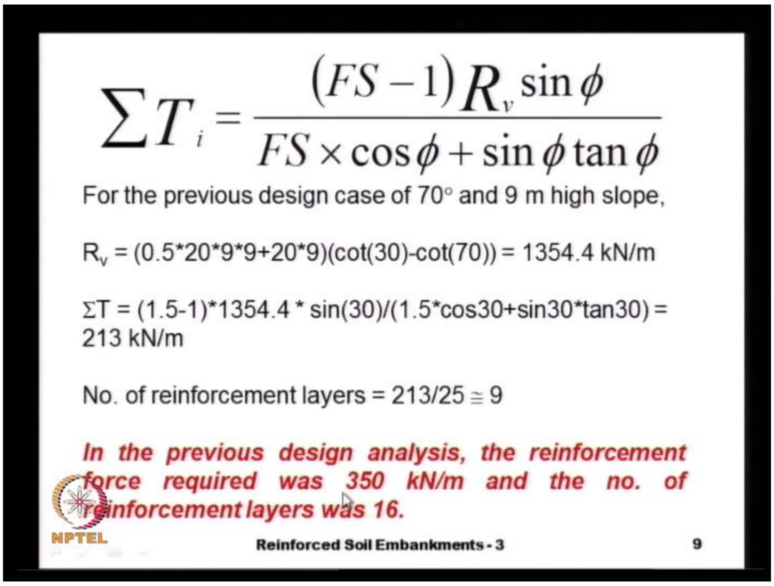
Well, actually we can illustrate this by looking at the previous case of the planar rupture surface, and let us revisit that analysis by considering the extra contribution, because of the normal and shear components. And just recap in the previous case of the planar wedge analysis we have considered the factor of safety as the shear resistance divided by the shear force, and the shear resistance was because of the normal component R v times cosine phi. And as you you may recall or we the the total vertical force, because of the weight of the soil within that wedge plus the the any contribution of the surchargers and the. So, on R v cosine phi times tan phi that is the shear resistance plus sigma T i times cosine phi that is the extra shear resistance that we have because of the action of the reinforcement layers a divided by the shear force. That is the R v sin phi that is the downward component, because of the soil weight and the external surcharge loads and so on. Means our factor of safety r v sin phi if we take product of cosine phi and tan phi we get sin phi plus sigma. And T i cosine phi divided by R v sin phi; that is 1 plus sigma T i by R v tan phi that is, if we do not provide any reinforcement layers the factor of safety that rupture surface is . Because that is basically the the plane of angle of repose and now let us modify the previous analysis by considering the additional components because of the normal and shear components.

And let us at consider the sigma T i as in the denominator. In the previous case we have considered in the numerator by considering the tangential component as directly adding with the shear resistance forces, we can also consider that those forces are opposing the

the shear forces. So, they reduce the active shear force that is trying to cause the failure of the slope. So, here the factor of safety has the same definition as in the earlier case that is the shear resistance divided by the shear force, that is R v cosine phi plus sigma T i cosine phi that is the normal component. This whole thing multiplied by tan phi that is tan phi is the our friction factor of or the shear resistance factor that divided by R v sin phi.

That is the downward acting component of the of the shear force, and minus sigma T i cosine phi that is the the component of the reinforcement force acting of the slope; that is opposing the action of the shear forces. So, our the factor of safety comes out as R v sin phi that is the cosine phi times tan phi plus T i sin phi times tan phi divided by R v sin phi minus sigma T i cosine phi, and this is our factor of safety and if we do not have any contribution from the reinforcements. The second terms in both the denominator and numerator they cancel out and we are left with a factor of safety of one as in the previous case, and because of the additional contribution of the reinforcement forces our factor of safety increases. And so a we can is actually by rewriting this equation in terms of the factor of safety.

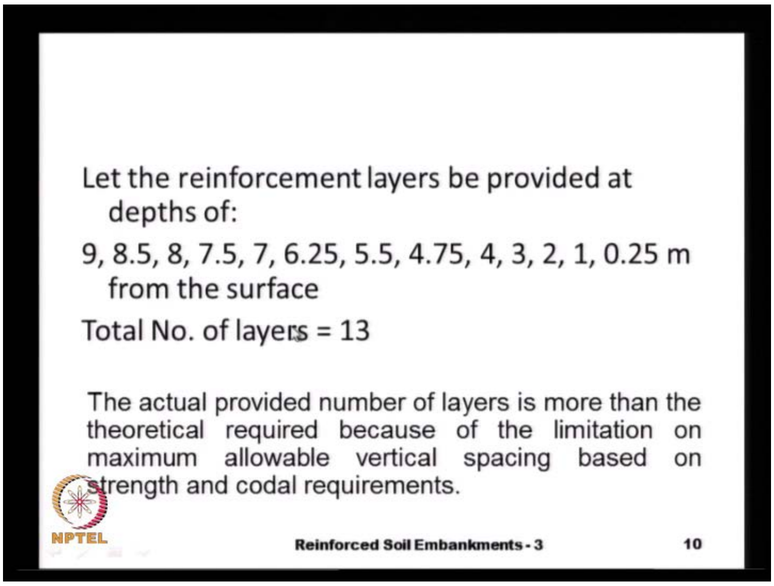
(Refer Slide Time: 13:55)



And R v we can determine the the the tensile force that we need to provide by placing the reinforcement layers to achieve a target factor of safety of F S like this, F S minus 1 times R v sin phi this whole thing divided by F S cosine phi plus sin phi times tan phi.

And let us calculate the quantity of reinforcement that we need to provide for the earlier case of 70 degrees slope, and 9 meter height the R v is one half of the gamma nine square; that is height plus 20 times 9 that is the 20 is the uniform surcharge times cotangent phi minus cotangent beta. That is the horizontal length of the of the slope on the wedge that comes out as 1354 kilo newtons per meter, and if we substitute this R v and F S of 1.5 and phi of 30 degrees in this equation. We get a sigma T as 213 kilo newtons per meter and in terms of the number of layers we require 213 by 25 that is 9. And if where if we compare this with the previous analysis in the previous case as you may recall reinforcement force required was 350 kilo newtons per meter, and the number of reinforcement layers was 16. So, we can say that is almost you are provided 50 percent more quantity of reinforcement in the previous case because of our assumptions.

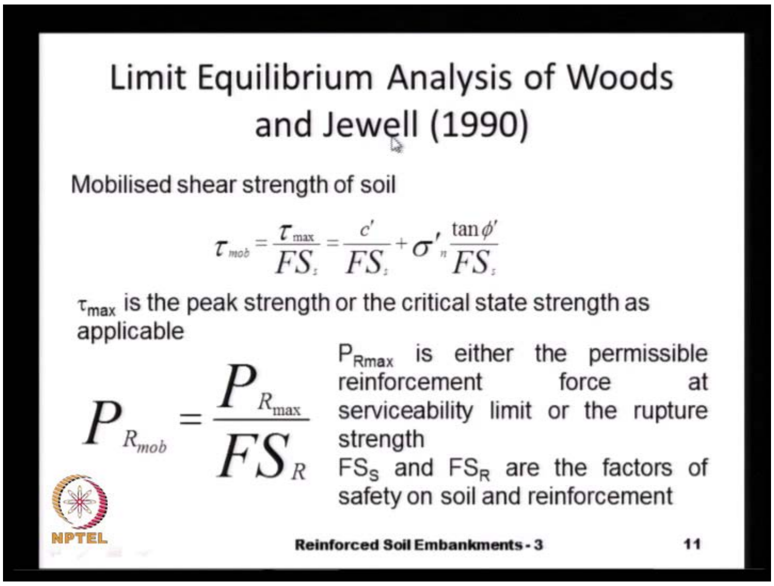
(Refer Slide Time: 15:49)



So, this the the quantity of reinforcement once again, it can be distributed over the height of the over the height of this the slope in this manner, we can provide one layer at the bottom that is at 9 meters depth another at 8.58 then the… So, on and just illustrate towards the top the reinforcement layers were provided at one meter vertical spacing in variably 1 meter vertical spacing is a more or less the maximum allowable spacing both in the b s code and also the federal highway administration code, and so the maximum spacing provided was 1 meter.

So, the total number of reinforcement layers required is 13, and the actual number of reinforcement layers provided is more than the theoretical theoretically required number of layers mainly, because of the limitation on the maximum allowable vertical spacing based on the strength requirements and also the codal requirements.

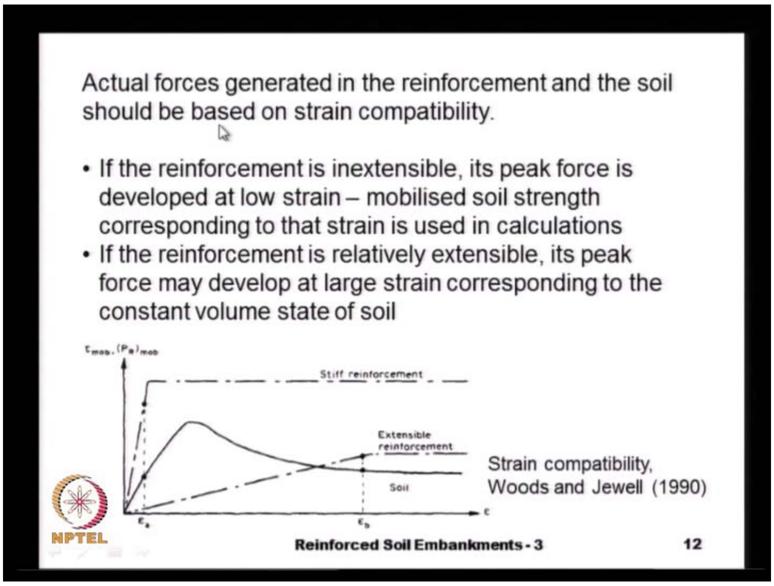
(Refer Slide Time: 17:02)



And now let us come back to the limit equilibrium analysis proposed by by woods and Jewell, the main consideration in all reinforced soil structures is the mobilization of the shear strength and he soils. And then mobilization of reinforcement force and for this purpose they have assumed. Some factor of safety F S with a subscript s as the factor of safety R factor to determine the mobilized shear strength of the soil this toe mobilized is equal to toe max divided by F S is equal to C prime by F S plus sigma N tan phi by F S. And the toe max is either the peak strength of the soil of the critical state shear strength as applicable, because now we need to also consider not only the soil, but also the reinforcement. And how compatible is the reinforcement with the soil and for estimating the reinforcement force the P R mobilized that is the mobilized reinforcement force is equal to P R max divided by the F S R where F S R is the is the factor of safety of the reinforcement layers.

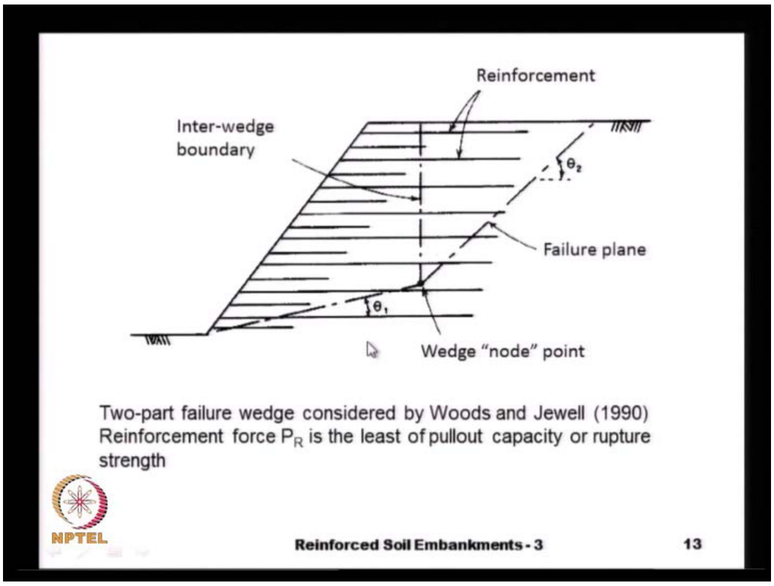
And The P R max is either the permissible reinforcement force at serviceability limit state or the rupture strength the usually the serviceability limit state happens at a very low deformation, whereas the ultimate limit state is that happens at a very large strain, where the the strength of the soil reaches the limit. And the and the reinforcement also reaches the rupture limit and this F S S and F S R are the factors of safety on soil and , because the strength of the soil. And the strength of the reinforcement may develop at different strain levels, because as we know the soil depending on the over consolidation or the denseness or the lose or the lose particle structure arrangement or the strain may develop at a different strain levels. And in the same way the reinforcement layers if we employee steel reinforcement it provide it develops its peak strength at a very low strain where as if we use some very soft geogrids may develop their peak capacity at a very large strain level.

(Refer Slide Time: 19:47)



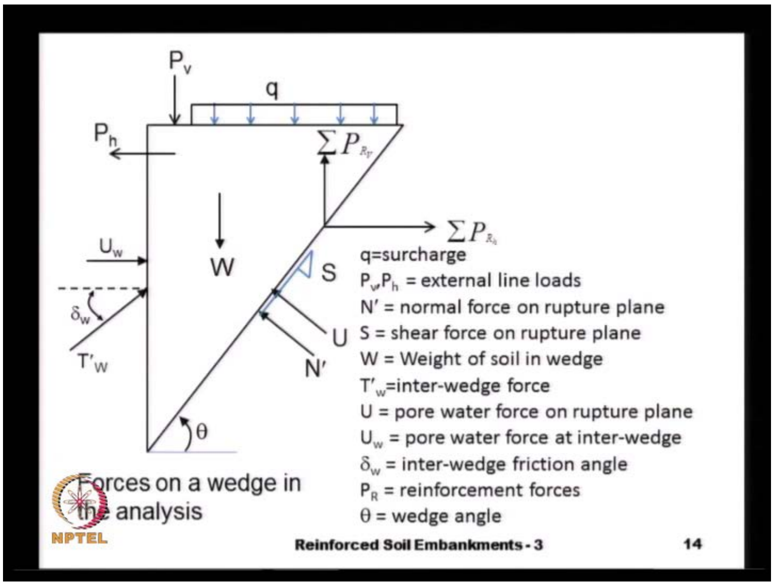
And so the woods and Jewell they postulate that the actual pore that we use for design purpose should be compatible based on the strain compatibility, see if the reinforcement is inextensible, its peak force is developed at a very low strain. Like for example, here we have a very stiff reinforcement, it develops the peak force at a very low strain. And the mobilized shear strength of the soil should be corresponding to that particular strain like for example, if we limit the reinforcement force this level based on some some other factors like the importance factor and so on. The corresponding strain is absalon a and the the shear strength or the shear stress in the soil only corresponding to the strain should be considered for our calculation purposes. Whereas if we use a very soft extensible geogrid the reinforcement may develops its peak capacity at about Absalom b. And that may correspond to limits and the the constant volume state of the soil. So, this is how we need to consider either based on the spectator strain levels the reinforcement force, and also the and also the the corresponding a soil stresses, and this they call as a strain compatibility.

(Refer Slide Time: 21:20)



And the two-part wedge that was considered by woods and Jewell is like this instead of considering one single rupture surface, they have considered two different rupture surfaces. And they explain that the two-part wedge method of analysis is good for steep slopes resting on on a trans foundation soil, and as we have seen earlier the two-part wedge is a close approximation for the slip circle; that we have passing through the toe and the woods and jewell they illustrate. That if you have a deep seated failure and the two-part wedge does not describe the the rupture surface well, and we should not consider the two-part wedge method of analysis for embankment resisting on on soft foundation soils.

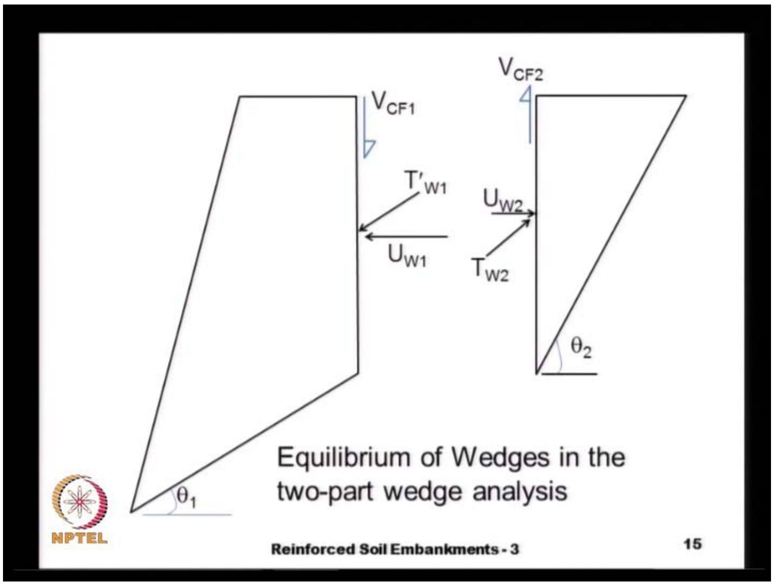
(Refer Slide Time: 22:19)



And these are all the different forces, that they have considered in the equilibrium analysis this is a typical wedge at an angle of theta, and this this wedge may be acting under its own self weight w or some external uniform surcharge q. And there could be some line loads either vertical or horizontal that is the P v and P h and the N prime is the is the normal force; that is acting on the on the rupture surface and the U that is the pore pressure force. And S is the the shear force on the rupture plane or the shear resistance sigma P R with a subscript h and sigma P R with a subscript v could be the the components of the reinforcement forces and the horizontal direction.

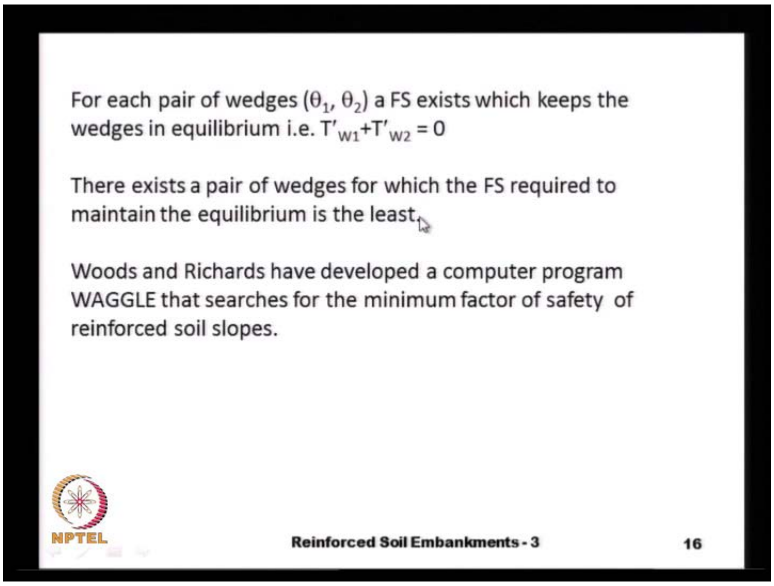
And also the vertical direction and then on this vertical surface, we could have a pore pressure force just as how we have pore pressure force on the on the rupture surface U U w is the the pore pressure force acting on the on the vertical surface. And then this T w prime is the the the force that is acting on the inter-wedge boundary, and it does not act normal to this, because of the because of the shear force that is developed here it may act at an angle of delta W, that is called as the inter-wedge friction angle and. In fact, the delta W is one of the variables at the Jewell has considered in his analysis and that we will see a bit later on.

(Refer Slide Time: 24:09)



And these are the two wedges that that we have the equilibrium of this wedge and this wedge together, and wedge is at an angle of theta one. And this is at an angle of theta two and these are the V C F is the is the cohesive force acting along the along the the inter-wedge boundary, and T W 1 and T W 2; these are the two inter-wedge normal forces. And is actually they act at an angle delta, because of the the inter-wedge frictional force and U W 1 and U W 2 are the pore water forces that are acting on the two wedges.

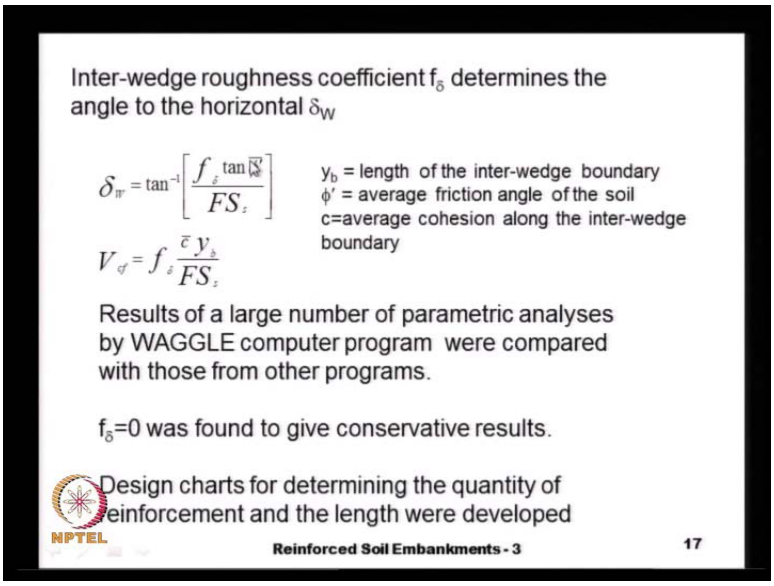
(Refer Slide Time: 24:53)



And the woods and Jewell they have considered very large number of these these rupture surfaces is actually here the variables are theta 1 theta 2. And then height of this node point wedge point from the base, and these three, once we define theta 1 theta 2. And y the height of the the node point from the base of the slope the entire geometry of the of the slip surface is defined, and they have considered very large number of these surfaces and they concluded that there exists a for a given wedge. And the two-part wedge there exist a factor of safety that keeps the the entire the both the wedges in equilibrium that is the T W 1 plus T W 2 is exactly equal to 0, and that we can determine by trial and error for a given a for a given geometry. And also there exists a pair of wedges for which the factor of safety required to maintain the equilibrium is the least that is just as how we aim for finding the least factor of safety for they through the slip circle analysis.

We can also do a similar thing for this for this two-part wedge method of analysis, and they developed a computer program called as waggle that searches for the minimum factor of safety of the reinforced soil slopes by considering the reinforcement force. And based on the on the rupture strength and the pullout force we can consider or we can determine the the critical reinforcement force that is contributed and these calculations are exactly as how we considered in the previous analysis. So, I am not repeating those calculations here, and so this program this computer program waggle searches for the minimum factor of safety for a given geometry the height of the slope and then the the slope angle and then the reinforcement configuration.

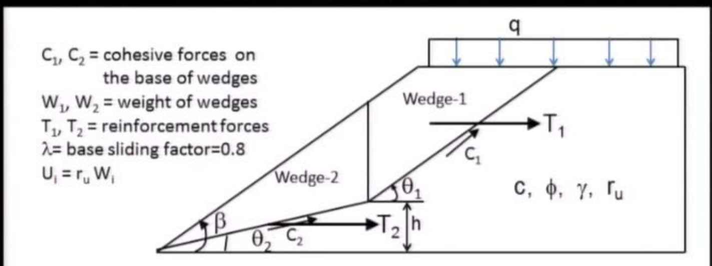
(Refer Slide Time: 27:16)



And in these analysis the inter-wedge roughness coefficient delta f w f delta is was found to be variable, and this f delta the delta w is the is the inter-wedge of friction angle can be determined as tan inverse of f f delta tan phi by F S of soil. And where in our the and then our the cohesive force V c of can be determined like this as f delta c bar times y b by f s where y b is the length of the inter-wedge boundary; that is the surface this is the inter-wedge boundary that is the boundary the vertical boundary between the two wedges. And F S soil F S S is the is the factor of safety in the soil and C is the average cohesion along the inter-wedge boundary this program waggle the f analyzed very large number of slopes.

And the results were compared with results from other programs other programs based on the slip circle analysis and what they found out that for the results that were obtained by assuming f delta is 0 they have obtained the most conservative results and. So, they proposed that it is best to neglect the f delta effect in this two-part wedge method of analysis. So, that we get more conservative design and that is more conservative means safer designs, and based on very large number of these these analysis they proposed some design charts for determining the quantity of reinforcement and the length.

(Refer Slide time: 29:14)



$$T_{tot} = T_1 + T_2 = \left[\frac{(W_1+Q_1)(\tan\theta_1 - \tan\phi_1') + (U_1\tan\phi_1' - C_1)/\cos\theta_1}{(1+\tan\theta_1\tan\phi_1')}\right]$$

$$+\left[\frac{(W_2+Q_2)(\tan\theta_2 - \lambda\tan\phi_2') + (U_2\tan\phi_2' - C_2)/\cos\theta_2}{(1+\lambda\tan\theta_2\tan\phi_2')}\right]$$

Effect of cohesion is to reduce the reinforcement requirement. Pore water forces increase the reinforcement requirement.

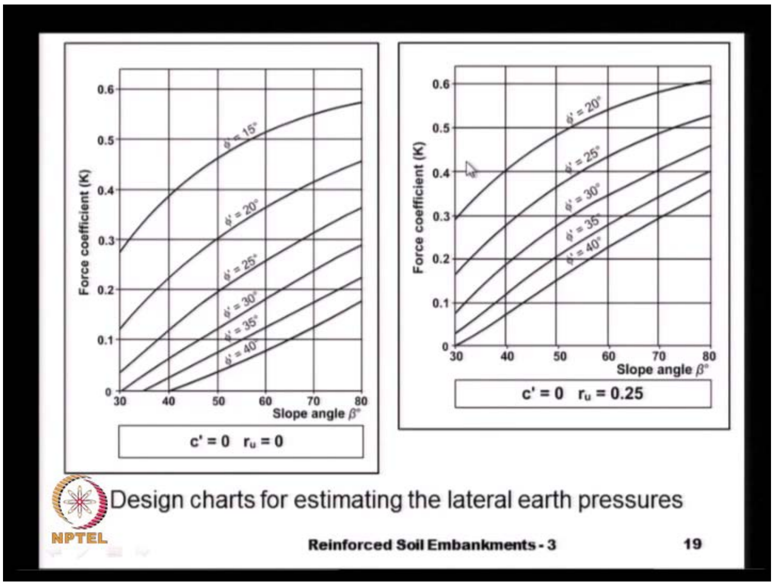
Reinforced Soil Embankments - 3 18

And the result can we summarized like this. So, we have two wedges wedge 1 and wedge 2, and the wedge 1 is at an angle of theta 1, and wedge 2 is at an angle of theta 2. And the slope itself is at an angle of beta, and we may have a uniform surcharge Q and then some other line loads acting. And the T 1 and T 2 are the reinforcement forces that is the minimum of the sum of the mobilized reinforcement force is at the minimum of the the rupture strength or the pullout capacity.

And let us say that C 1 and C 2 these are the cohesive forces mobilized along the base lengths, and let us say W 1 and W 2 are the are the weights of the soil within the wedge, and the Q 1 capital, Q 1 and capital Q 2. These are the surcharge forces that are acting on the surface of the wedge one and wedge two, and if this is... So, and then we can also write the pore pressure force u in terms of the bishop's pore pressure parameter r u times b and W i where W i is the is the vertical weight of the weight plus the surcharge force acting on on the respective wedges. And this the total quantity of the of the reinforcement quantity is T 1 plus T 2 where T 1 is the quantity in wedge 1 and T 2 is the quantity of reinforcement in wedge 2.

That is written as W 1 plus Q 1 time tan theta 1 minus tan phi 1 plus U 1 tan phi 1 minus C 1, that is the the cohesive force acting on the on the base of the wedge one divided by cosine theta 1 this whole thing divided by 1 plus tan theta 1 tan phi 1. And the force developed in the wedge 2 is W 2 plus Q 2 tan theta 2 minus lambda tan phi 2 and so on where lambda is at the base sliding factor and that is considered as 0.8, and as we can see the effect of cohesion is to reduce the reinforcement requirement. And the effect of pore water force is to increase the reinforcement requirement, because as we have higher pore water forces we can expect higher lateral force that are acting and that needs to be the resisted by our reinforcement layers.

(Refer Slide Time: 32:00)

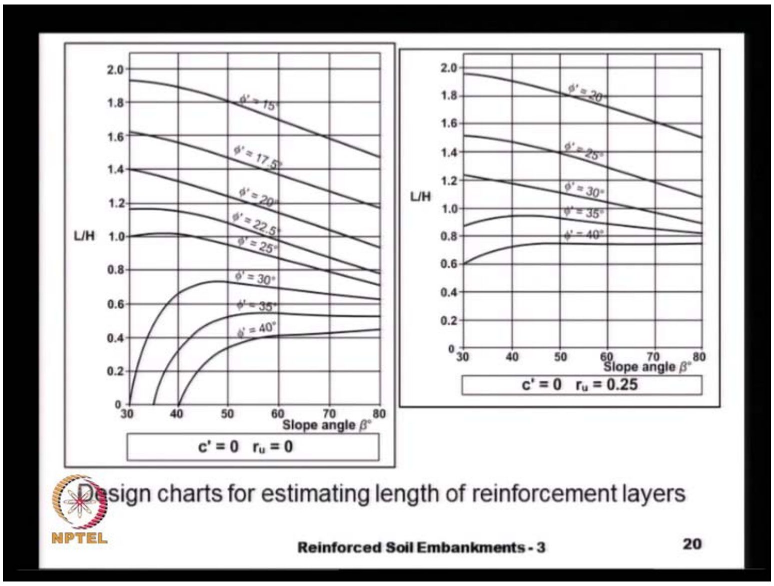


And the results from all these parametric analysis they were published as design charts like this here. For example, in this the chart on the left hand side we have the slope angle beta on the x axis ranging from 30 degrees to 80 degrees, and on the the y axis we have the force coefficient. And internally we have number of friction angles 15 degrees 20 degrees 20 30 35 40, and the y axis gives us the force coefficient K that we had earlier obtained using a formula that was given, and these results are given for two different pore pressure coefficient's r u of 0 and r u of 0.25.

And the effect of the pore water pressure is to increase the lateral pressure coefficient say for example, say for slope angle of 70 degrees. And and friction angle of 30 degrees the K factor for the dry soil is about 0.24 where as with the pore water pressure coefficient of 0.25 it is 70 degree slope, and then 30 degrees is the friction angle of the soil is nearly 0.4. So, there is a substantial increase in the the lateral pressure coefficient; that means, that our requirement of the reinforcement force is also much higher with with the pore pressure coefficient's.

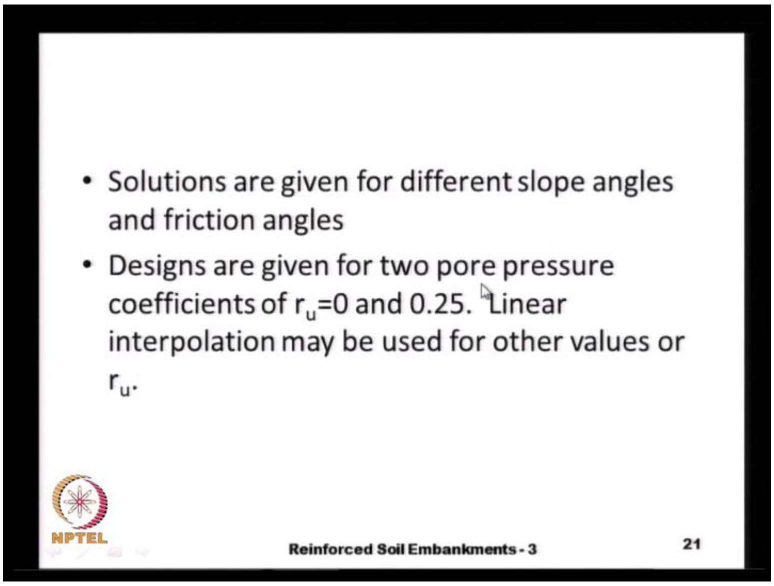
And If you have a pore pressure coefficient in between 0, and 0.25 we can do a linear interpolation and usually case where the pore pressure coefficient more than 0.25 does not arise, because in most cases. We use relatively good quality granular soil for construction of a steep embankments steep slopes are reinforced soil retaining walls. And if our r u is coming out as much higher than 0.25 becomes uneconomical to to come out with with safe designs that case we can change the type of soil.

(Refer Slide Time: 34:22)



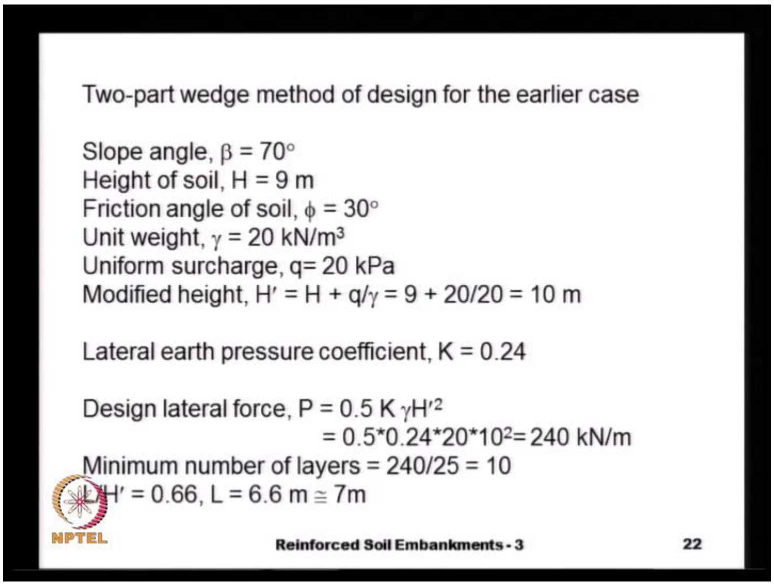
And the design charge for estimating the the reinforcement lengths are also provided provided like this and the length that we estimate from these design charts is sufficient not only prevent the base sliding at the base and also the reinforcement pullout and the the the most critical reinforcement layer. And once again we can see that these results are proposed for two different pore pressure coefficient's of r u 0 and 0.25 and once again let us see for 70 degree slope angle. And 30 degree friction angle our L by H is approximately 0.65 and if you see the case of port pressure our 70 degrees and 30 degrees is nearly 0.95. So, there is almost 20 to 30 percent increase in the length of reinforcement layers because of the the port pressure forces.

(Refer Slide Time: 35:33)



And these solutions are given for different slope angles, and then the friction angles and the designs are given for two different port pressure coefficients; that is r u of 0 and 0.25 and we can use linear interpolation between r u of 0 and r u of 0.25.

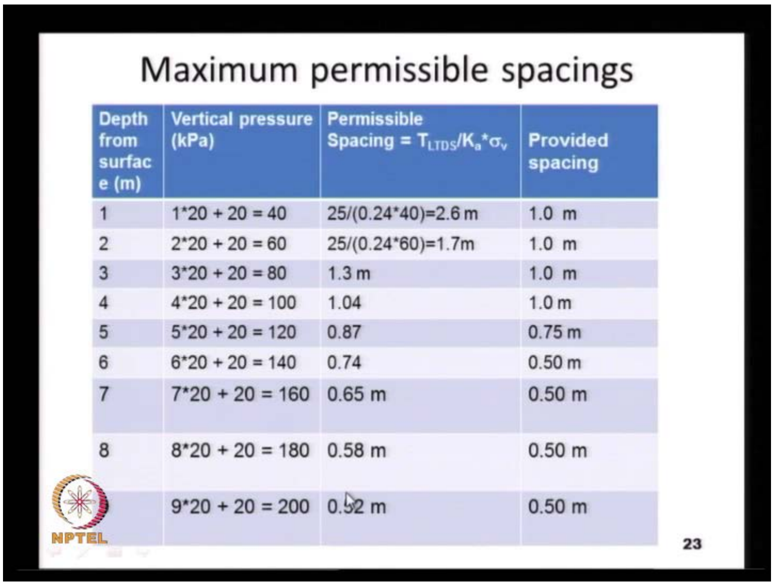
(Refer Slide time: 35:55)



And now let us calculate the solution for the same embankment that we have designed earlier that has a slope angle of 70 degrees, and the height of the slope height of a soil is 9 meters. And the friction angle of the soil was 30 degrees the unit weight is 20 kilo newtons per cubic meter and the uniform surcharge q is 20 k P a. And for applying these charge we get a modified height of the embankment H the H prime is H that is the the the soil height plus the q by gamma w q is the uniform surcharge that we have and gamma is doing the unit weight.

In this case our modified height of embankment if ten meters and our lateral pressure coefficient K is 0.24 for for 70 degree slope, and 30 degrees friction angle is approximately 0.24. And so our design lateral force is 0.5 K times gamma H prime square that works out to 240 kilo newtons per meter length of the embankment, and the minimum number of layers required is 240 by 25. That is approximately 10 and L by H prime is 0.66; that is once again going back to to these charts 70 degree slope and 30 degrees friction angle it is about 0.66. And our length of the reinforcement is 6.6 0.66 times 10 that is 6.6 meters, and we can round it off to 7 meters in the next higher integer 7 meters.

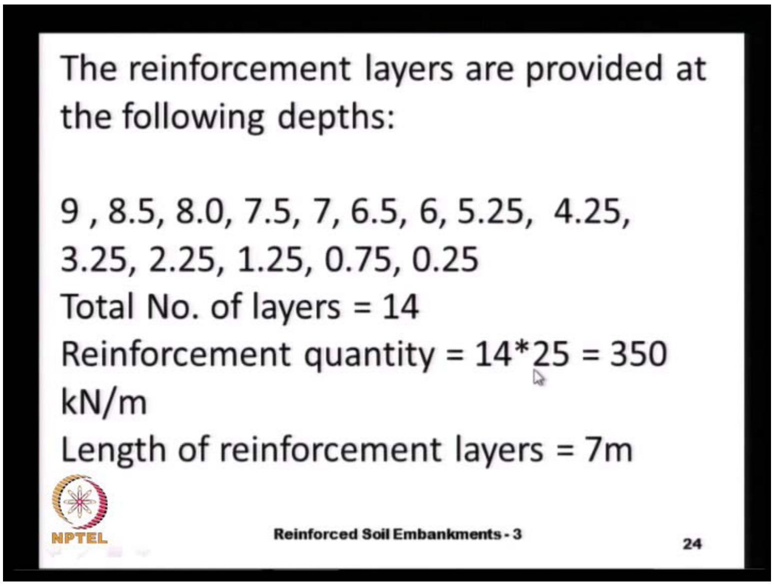
(Refer Slide Time: 37:54)


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/(0.24*40)=2.6 m	1.0 m
2	2*20 + 20 = 60	25/(0.24*60)=1.7m	1.0 m
3	3*20 + 20 = 80	1.3 m	1.0 m
4	4*20 + 20 = 100	1.04	1.0 m
5	5*20 + 20 = 120	0.87	0.75 m
6	6*20 + 20 = 140	0.74	0.50 m
7	7*20 + 20 = 160	0.65 m	0.50 m
8	8*20 + 20 = 180	0.58 m	0.50 m
	9*20 + 20 = 200	0.52 m	0.50 m

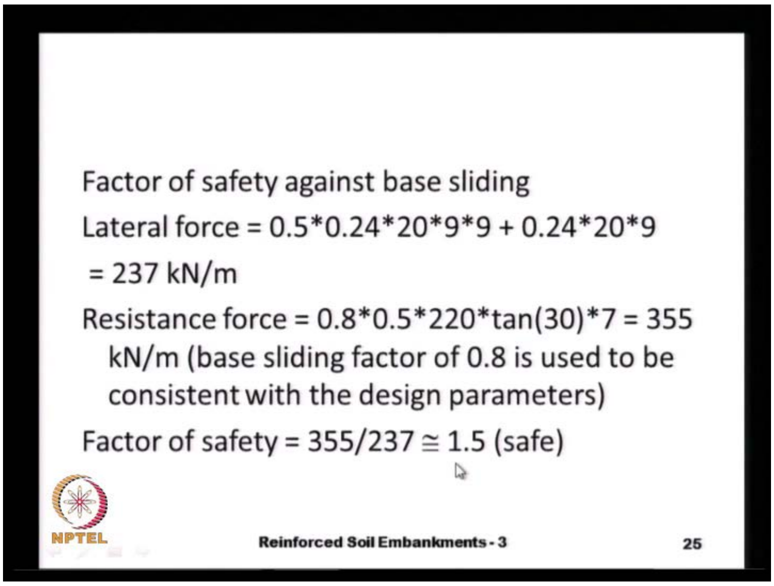
And this quantity of reinforcement of 240 kilo newtons per meter that should be be spread uniformly over the height of the the soil, and just as how we had done the calculations earlier. We can repeat this calculations based on the tensile strength of the reinforcement that is 25 kilo newtons per meter, and 0.24 is the lateral earth pressure coefficient and that depth of 1 meter our permissible reinforcement spacing comes out as 2.6 meters, but as per the cordial provisions we should not provide at a spacing more than 1 meter. So, at the bottom at 9 meters depth the spacing required is 0.52. And we round it off to the closest multiple of the the compaction layer thickness that is 250 m m times 2; that is 500 m m and up to 7 meters depth the required spacing is 0.65. And even at 6 meters depth the required spacing is 0.74 and the nearest 1 is 0.5 nearest lower multiple of the compaction layer is 0.5.

(Refer Slide Time: 39:15)



So, considering all these factors we provide reinforcement layers are depths of 9 meters 8.5 8 7.5 7 and so on, and totally the number of layers provided is 14 although we calculated that we require only 10 layers, but the actual numbers provided is 14 and the reinforcement quantities 14 times 25 that is 350 kilo newtons per meter. And the length of the reinforcement layers is 7 meters is actually in terms of the design that we have obtained with planer rapture surface and the path wedge method is not. So, much in the quantity of reinforcement layers provided, but in terms of the length of the reinforcement layers.

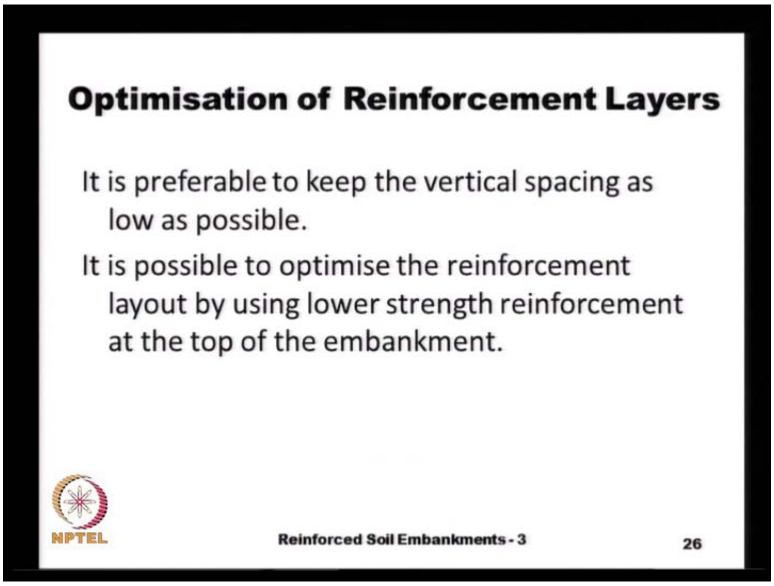
(Refer Slide Time: 40:02)



And the factor of safety against base sliding the lateral force is 237 kilo newtons per meter because our lateral pressure coefficient is 0.24. So, we can calculate this and the resistance force against the against base sliding is 0.8 times 0.5, that is times 220 that is the average vertical pressure at the base of the embankment times tan 30.Tthat is the friction factor times 7 is the length of the reinforcement layers that is 355. And here we have used base sliding factor of 0.8, because that was what was assumed for for developing these designs arts the designs arts for calculating the length of the reinforcement were based on the lambda factor of 0.8. So, our factor of safety comes out as exactly 1.5.

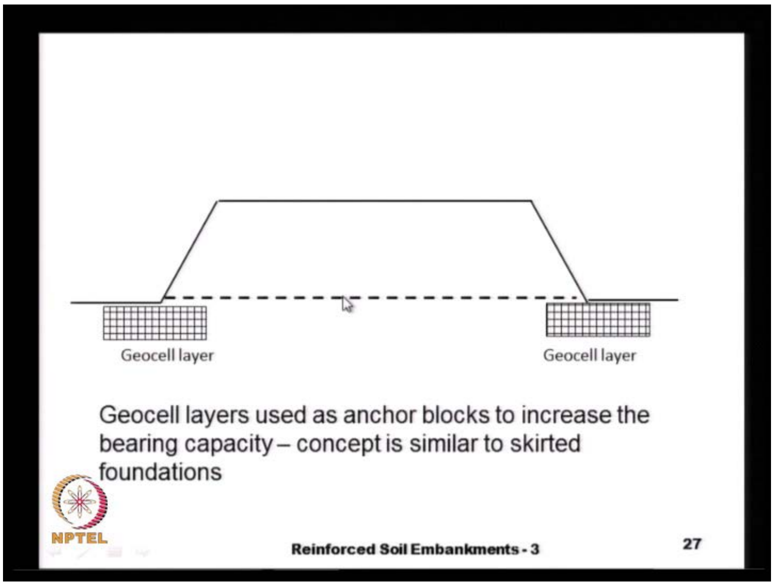
So, it is safe and so our design based on length of 17 meters, and and then total quantity of reinforcement force of 350 kilo newtons with 14 layers is safe against rapture surface. And then the pullout considerations the actually how economical e are these design sections that we have provided is actually we can optimize the reinforcement layers, because as we see here the maximum permissible spacing for this strength of reinforcement is 25 is 2.6 meters. Because our cordial requirements suggest the maximum spacing as 1 meter we have provided at 1 meter spacing and so we can, so some type of optimization.

(Refer Slide Time: 42:04)



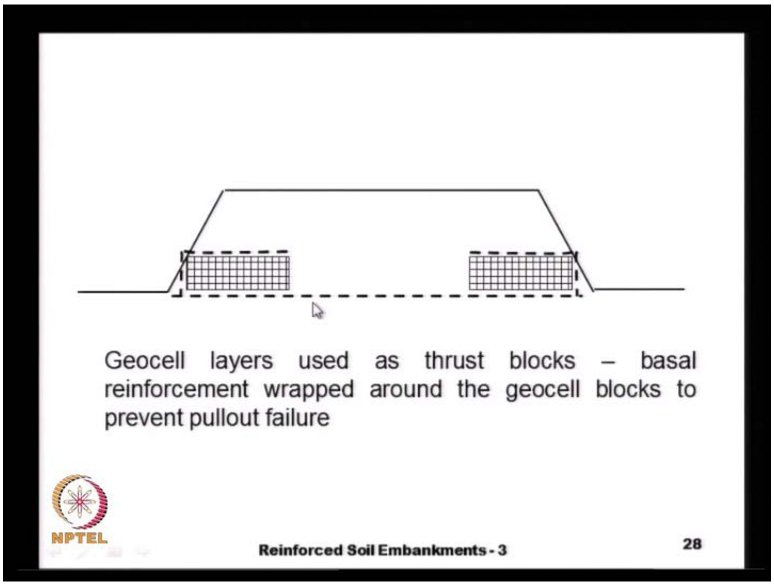
And to come out with better reinforcement spacings because it is preferable to use lower strength reinforcements at the near to the top of the soil soil slope. So, that we can maintain an economical vertical spacing and it is also good to keep the vertical spacings as low as possible instead of using one very high strength in geogrid or a geotextile. It is better to use larger number of reinforcement layers the spread over uniformly over the entire height of the soil to to come out with better designs. And the concept here is the same very similar to reinforced concrete design where in the quantity of steel is distributed by by providing smaller diameter bars rather than providing one just one very large diameter steel bar.

(Refer Slide Time: 43:07)



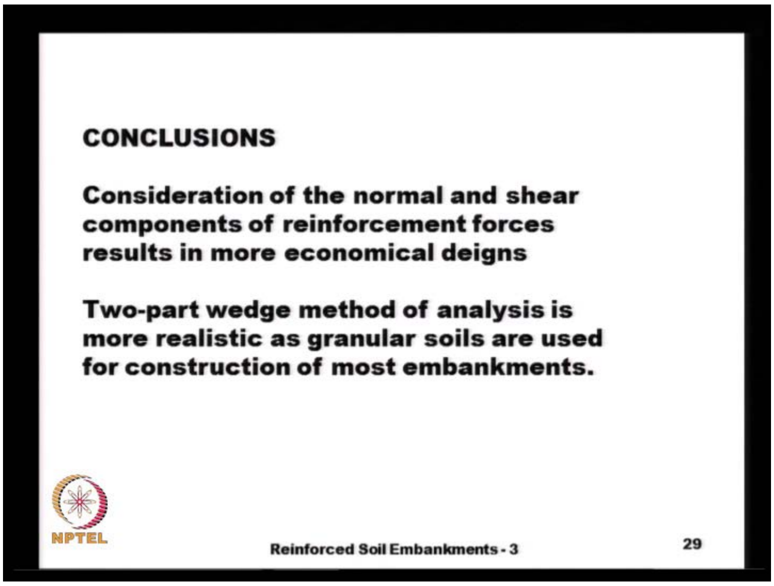
See in the earlier case we have considered the design of basal reinforcement for for very soft soil foundations, and sometimes it may happen that our reinforcement that is provided is not adequate. In that case we can have we can employ a concept very similar to this cutted foundations by providing geocell layers and the two edges of the slope. So, that the the thrust that is acted upon by by the embankment is resisted by these geocell layers because the geocell layers are completely buried in the soil. And so there is large lateral pressure is developed against this foundation soil, and because of this the strength of the reinforcement layers increase.

(Refer Slide Time: 44:06)



And the geocell can be provided in two manners one is like this or or we can also provide the geocell layers within the body of embankment. In that case we can rap the the the basal reinforcement around the geocell layers. So, that these two geocell bodies they act as more like a thrust blocks or the to resist the lateral thrust.

(Refer Slide Time: 44:35)



And just to highlight we have seen that considering the normal, and shear components of the reinforcement forces results in more economical designs. And that is employed in in our designing or developing design charts for the for the two path wedge method of analysis, and the two pat wedge method analysis based on planar rapture surface is more realistic per granular soils which are mostly used for our construction purposes. Thank you very much.

(Refer Slide Time: 45:09)

And if you have questions you can e mail me at this e mail address.

Thank you very much.