Geosynthetics and Reinforced Soil Structures
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Lecture - 26
Geosynthetic Reinforced Pile Platforms for High Embankments

Very good morning students, let us continue our discussion on the construction on extremely soft clays.

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One option is the use of Geosynthetic Reinforced Pile Embankments, and let us see what they are and how we employ them for construction in this lecture.

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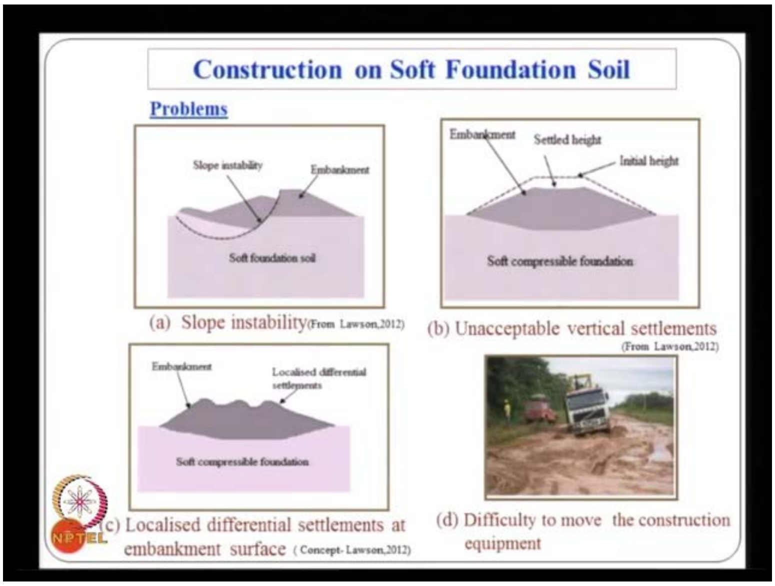

These are all some of the problems that we can encounter in the case of construction on soft foundation soils, we may have a deep seated failure like this or excessive settlements or localized settlements or even the difficulty in moving our construction equipments. So, you will have to deal with all these problems in the case of soft clay soils.

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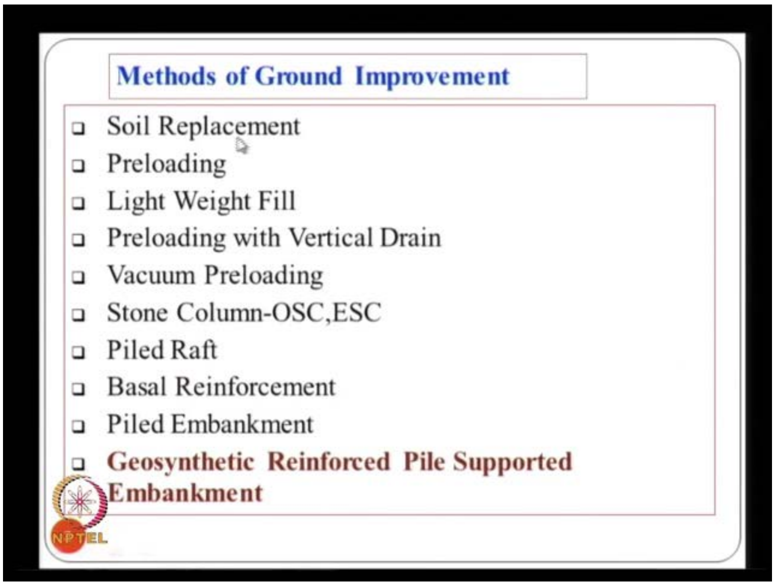

And we have already seen that, there are several options, that are available for construction on a soft clays, that is the easiest one is the soil replacement, if the thickness of the clay soil is very small. We can think of replacing or the preloading that we have seen in earlier lectures either by surcharge consolidation or by vacuum consolidation, and using the light weight fills like polystyrene or pond ash and so on. And the most recent one of all these techniques is the geosynthetic reinforced pile supported embankments.

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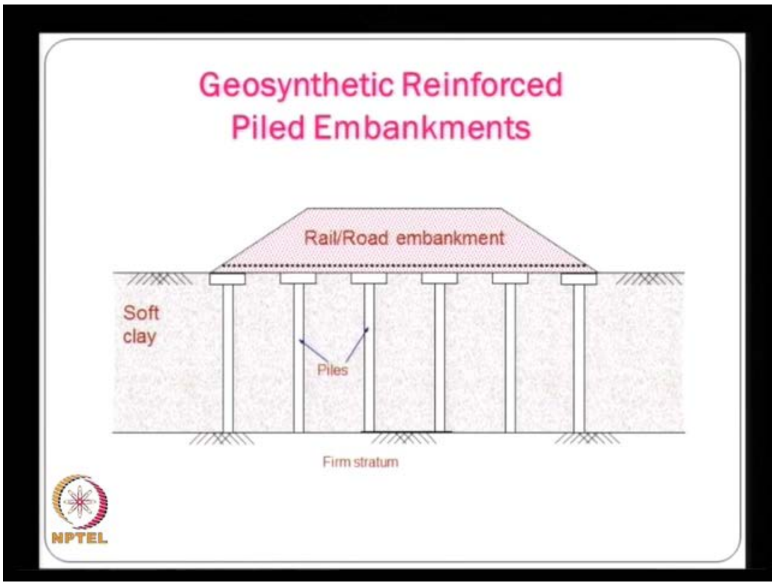

And this has actually become very popular especially for very high embankments, and for very thick clay soil deposits. Conceptually, the geosynthetic piled platforms are like this. And the concept wise it is similar to pile raft, where we have a solid raft reinforced concrete raft and the piles except here. We do not use any steel reinforcement, all these piles are small diameter piles, which are installed to bearing stratum. And all these are mostly plane concrete, they are not reinforced with steel then they may have a small pile cap, and then we will have a sufficient amount of reinforcement the geosynthetic reinforcement.

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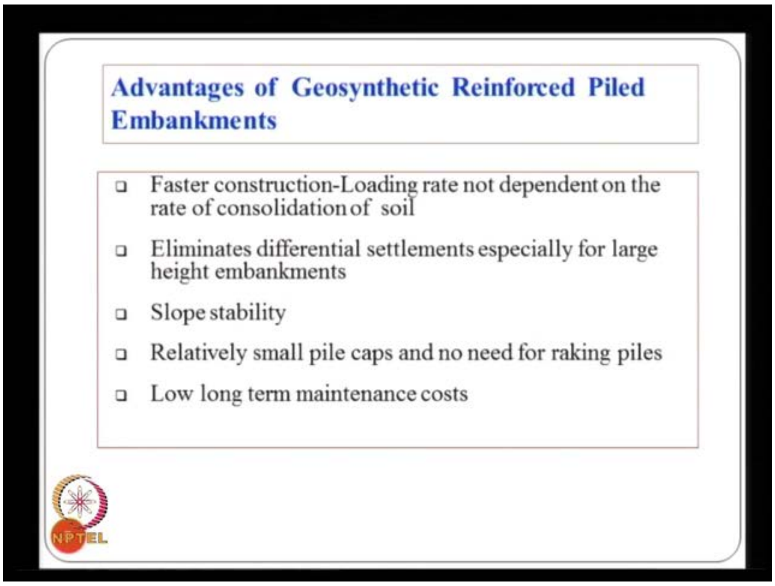

And some of the advantages that, we have with this type of construction is we can have a faster construction and the loading rate is not dependent on the rate of consolidation of soil. Because, now we are not depending on the on the strength of the soil, but rather on the strength of the whole system with piles and then the and platform that, we have made of a geosynthetics. And it eliminates the differential settlements, especially for very high large height embankments and we do not have to deal with the slope stability problems, because the pr the load is taken by the relatively rigid system of the piles. And geosynthetics and the relatively small pile caps and there is no need for raking of the piles.

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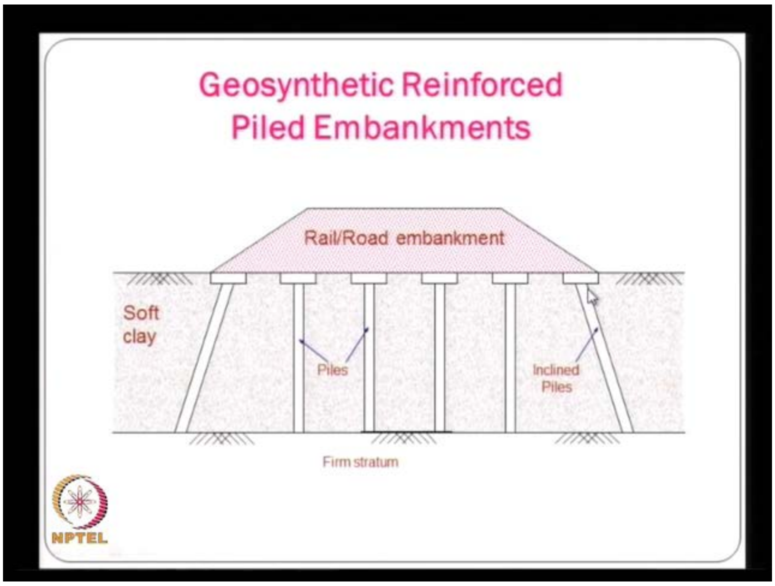

Especially, the pile raking is something at the end of the embankments, we give the inclined piles to take care of the lateral thrust from the high embankments. And because the provision of horizontal of layers of reinforcement all the lateral load is taken by the reinforcement, and because of that the lateral loads transferred into the piles or much smaller or very low and so we can just think of providing vertical piles.

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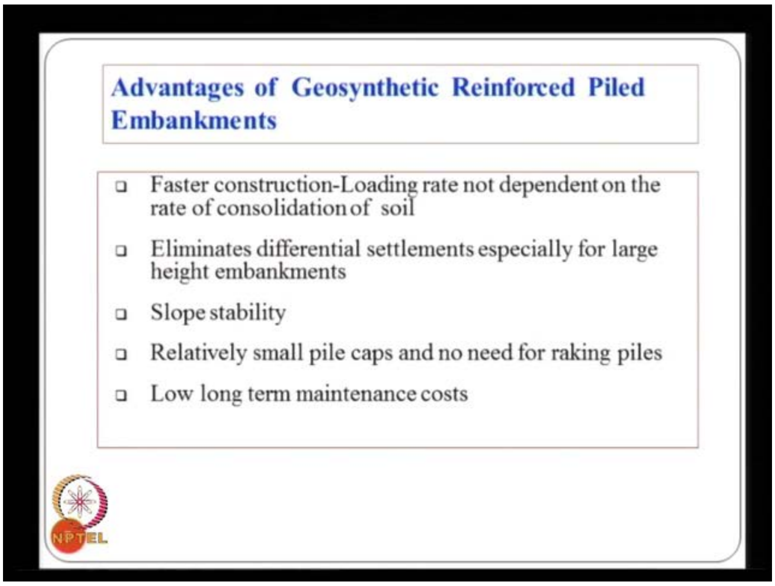

And is also, we will have very low maintenance costs, because the differential settlements on the total settlements, that are expected from the systems are small.

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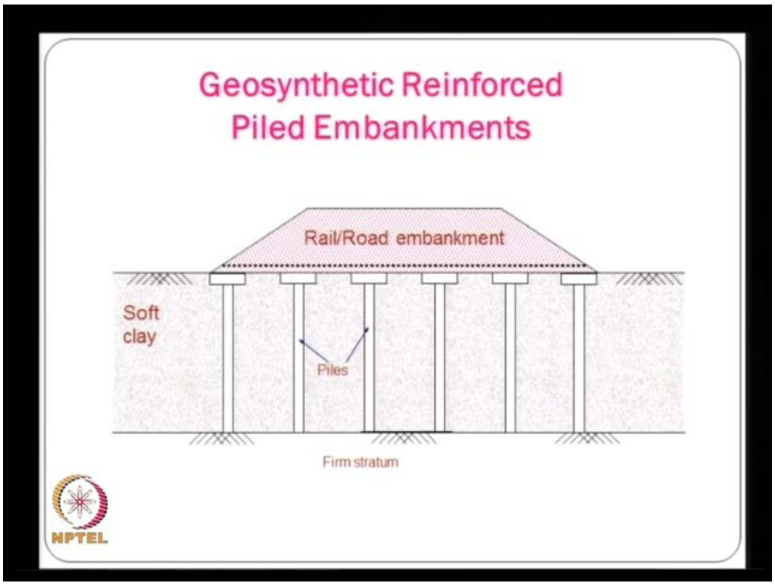

Basically, because these piles, they extend into the firm stratum and because of that the expected settlements both total and differential or very small with this type of constructions.

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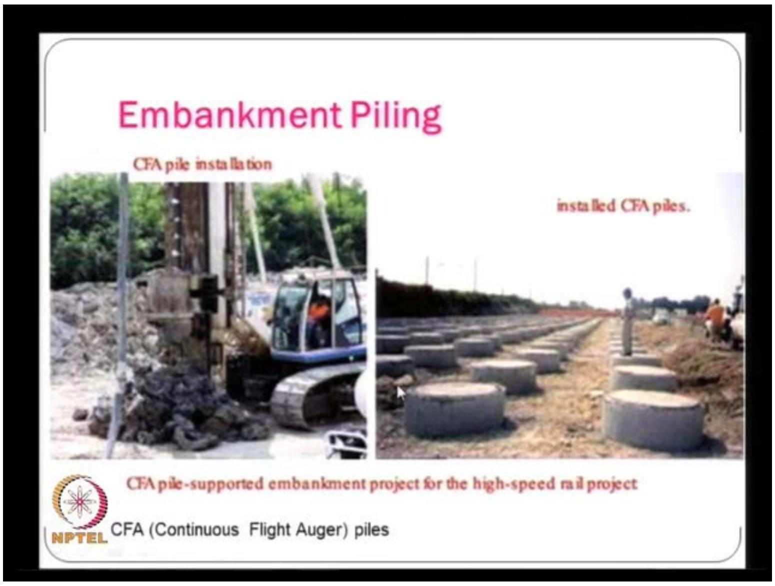

And let us see some of the examples here at one project, we see the castings to piles, we which are, which we call as a castings to piles or continuous flight auger. And on the left hand side, we see the bore well being made the right hand side the finished piles that are already in place.

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And here, we see another example of the same thing here, we have the pre-cast piles and then we can just simply push them into the soil depending on the type of soil, if the soil is very soft, it is easy to work with pre-cast piles and then push them into the ground. After that, we have a layer of geogrid and then a layer of geotextile is actually takes a combination of reinforcement and the separation. The separation is required, because we have a good quality aggregate and that should not mix up with the clay soil that is there in the foundation soil. So, here we have both geogrid and the geotextile.

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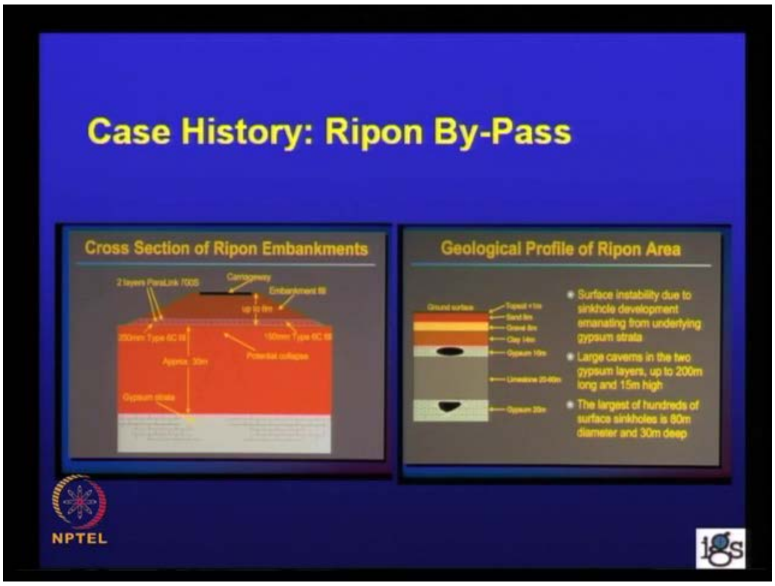

And another place where the geosynthetic piled embankments been constructed successfully was in the ripon by-pass, ripon area and the United Kingdom. It is a very treacherous area, because the foundation soil consists of very thick deposits of limestone or cost deposits, which are which is basically gypsum and here, there are many arterial wells that means, water is flowing at very high pressure.

And the gypsum or the limestone is a very soft material and once the water flows through at very fast pace, simply the limestone gets dissolved and flows along with water and that results in very large wides formed at the surface. So, the expected problems this place are the surface instability because of this sinkhole development or large caverns, because there are 2 gypsum layers and the caverns as long as 200 meters and a 15 meters high, they were observed in the past.

And the largest of the of the surface sinkholes about 80 meters ammeter and 30 meters deep. So, it is a and in this area whatever the road that we construct a road embankment, we should expect any future settlements or the sudden sinking of the entire ground. And so here it is best to transfer the loads to a deep stratum, which is not subjected to this sinkhole formations.

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And here, we see once again, the small diameter piles installed and then very strong geogrid.

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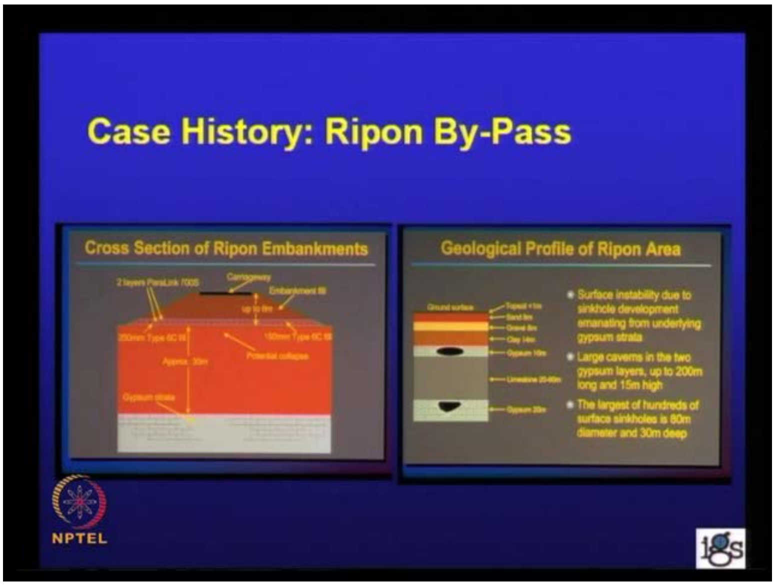

Is actually, this is this geogrid is called as paralink geogrid.

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Basically, we have number of these this polymeric strips, each may be about 50 kilo Newton to 100 kilo Newton capacity, they all are linked together.

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And another place where, these low transfer platforms have been successfully, used was in the duffryn link, once again we see this a small diameter piles that are pre-cost and pushed into the ground.

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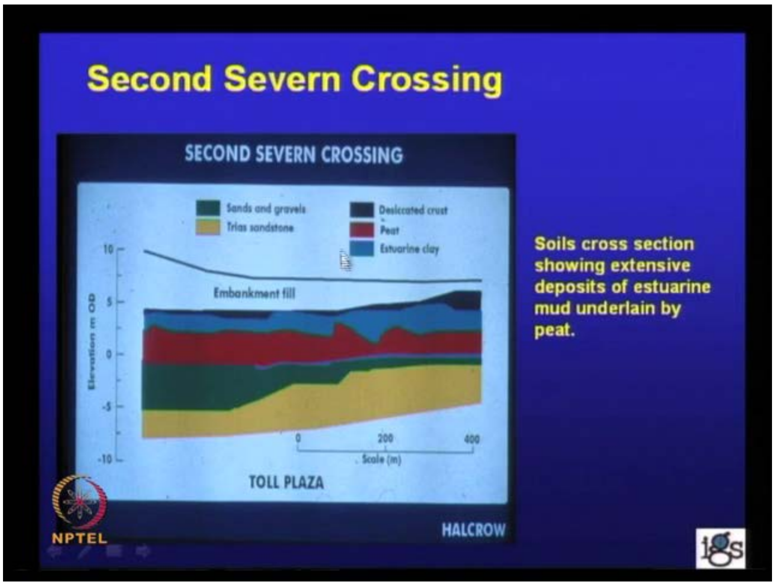

And the second severn crossing, it is once again through a very highly marshy area and we can see the soil deposit is actually it is a thick layer of P T soil peat and then estuarine clay with both of them are very very soft. And so the any construction of this is the embankment construction and if it passes through that type of a soil, we should expect very large settlements and then the problems, because of lateral flows.

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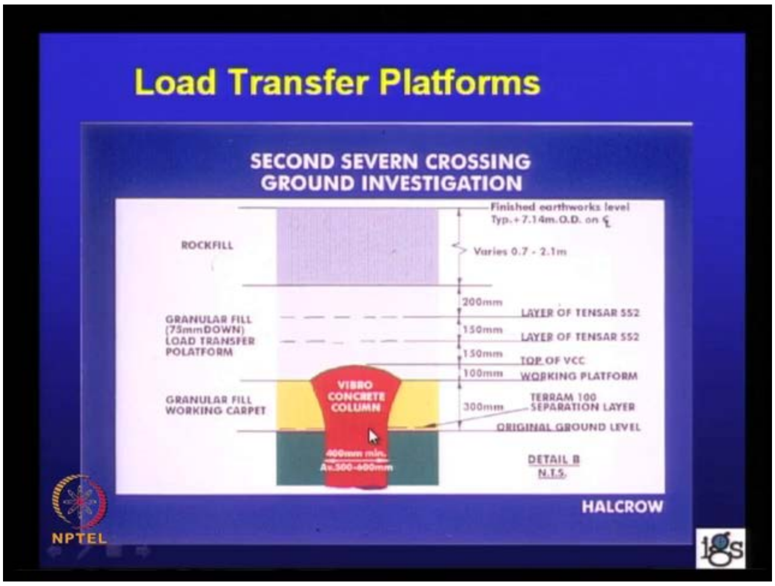

And the proposed solution was the placing the vibro concrete columns.

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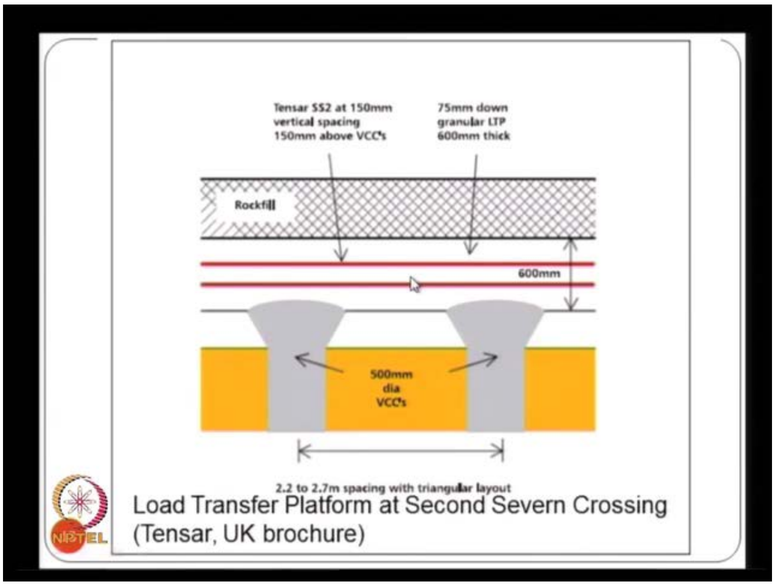

At about 2 to 2.7 meter spacing and the then that is reinforced with 2 layers of relatively weak geogrids S S 20 or S S 2, they have a tensile strength of about 20 kilo Newtons per meter.

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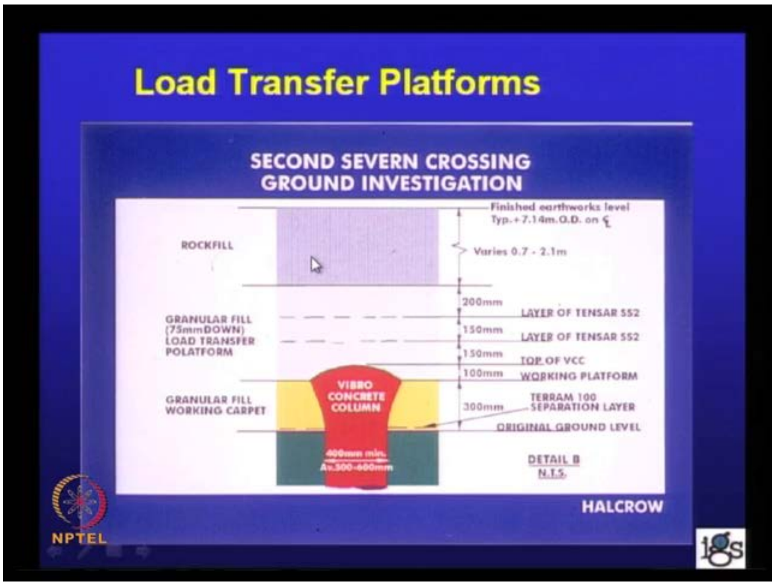

And the platform itself is constructed using 2 layers of geogrid and then rockfill anywhere from 0.72 metes thick.

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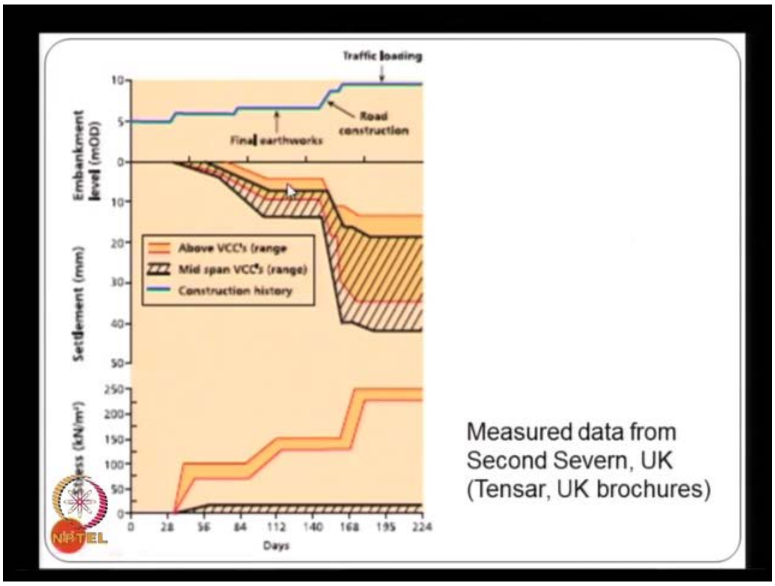

And the performance and the monitored performance is excellent as shown here and the bottom part, we see the stress and the top part, we have the settlement is actually the with time. The construction of embankment took place for constructing about 10 meters high embankment, it took almost 220 days is about 7 and half months, which is relatively past. And these are the settlements is actually above the V C C this red lines, they show the settlements on top of these piles and the mid span between the pile supports is anywhere from about 45 millimeters to 15 millimeters. So, the settlements in spite of the fact that.

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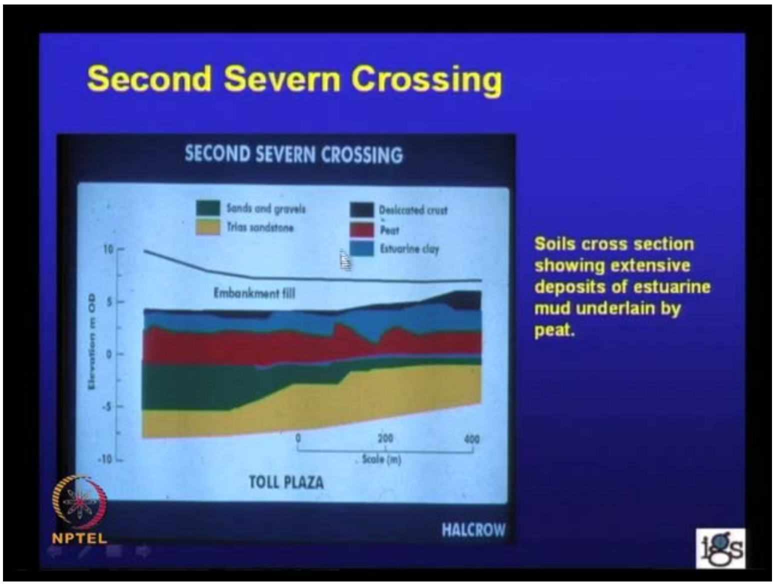

The thickness is very large the thickness of the soft clays ranging, anywhere from plus 5 to minus 5 about 10 meters soft clay deposit.

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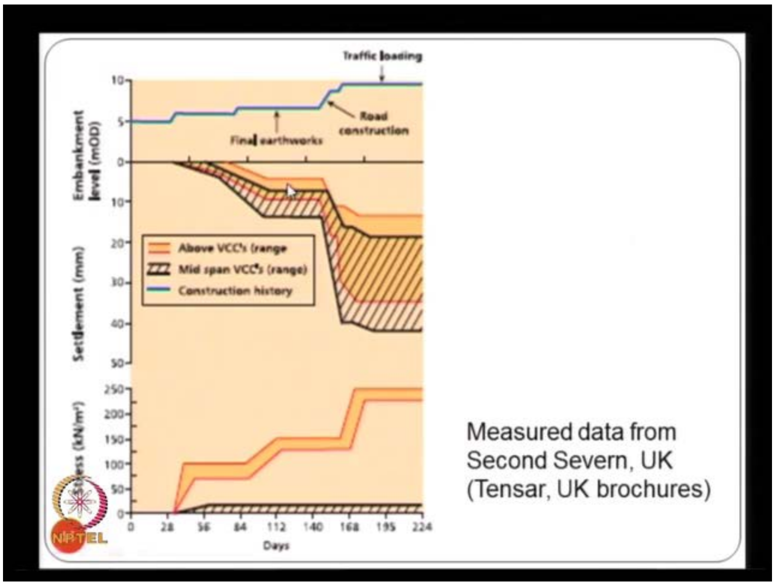

And more importantly the bottom graph, it shows the loads that are transferred into the subsoil and the piles. And the very small amount of applied load is transferred into the subsoil, this is the this shows the pressure transferred is hardly about may be 20 to 25 k P a whereas, most of the load is transferred into the piles almost 250 kilo pascals. So, the expected settlements are very small and the height of the embankment itself is 10 meters, so it is very high embankment.

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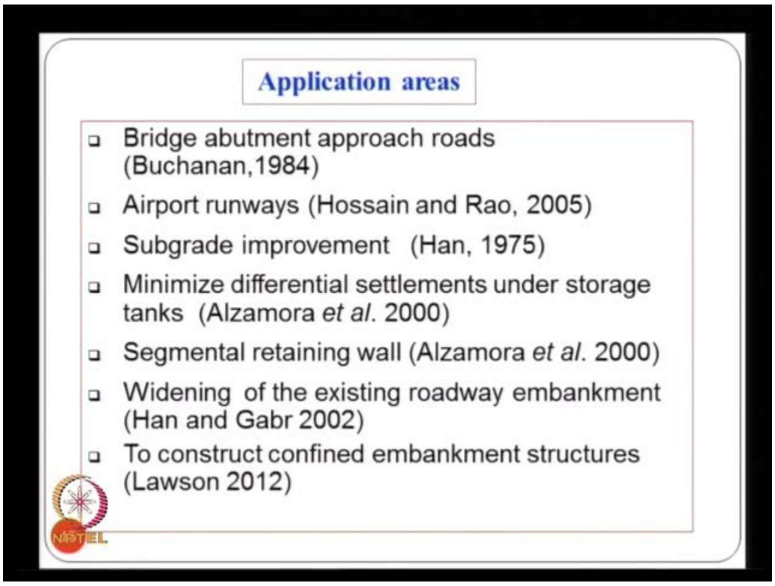

And what are some of the application areas for these geosynthetic pile platforms, the bridge abutment roads all the examples that we have seen earlier, they are all part of the approach embankments. Then airport runways and then subgrade improvement for construction of highways and minimize differential settlements under storage tanks, for example, we have many oil storage tanks all over India, which are mostly constructed on soft clay soils because most of the refineries there along the coast lin, where the soil is very soft. And this concept has been used even for supporting the storage tanks oil storage tanks, then the retaining walls then widening of the existing roadway embankments.

Especially, when we have a roadway, which is quite old say if it is there for about 1525 years, it should have undergone full consolidation settlements. Now, if we have to iden this embankment, the problem that we have to face is the old part of the embankment is not going to settle down. Whereas the new part is going to settle down, because of the new loading that are applying, in that case, we can think of supporting the widen part of the road embankment on this type of pile supports, and of course to construction of the tall embankment structures.

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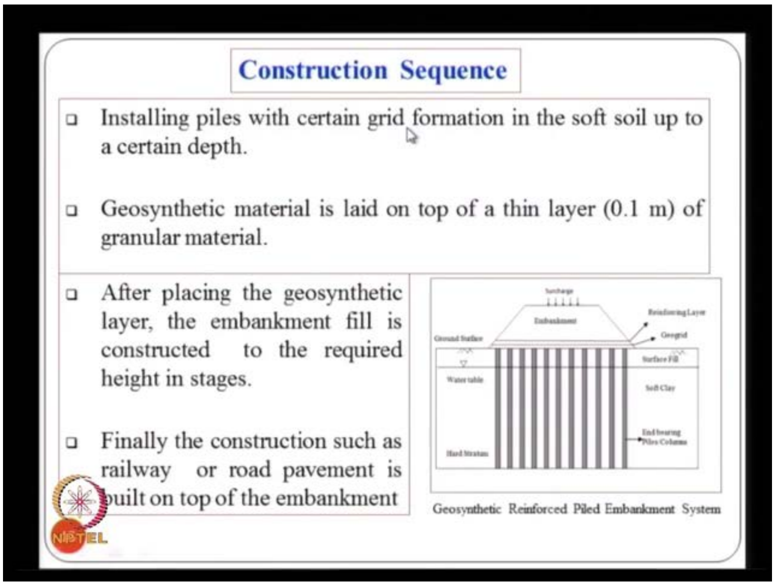

The construction sequence is very simple, we have the piles installed in the certain grid pattern, either square pattern or triangular pattern, that we have seen earlier, we can install this piles, we can either have a board castingsitu piles or driven or pre-cost piles. And then we lay the a geosynthetic layers with a thin cover of about 100 m m on top of the piles. So, that the geosynthetics are not directly in contact with the hard surface of the pile, because if that is so there is a possibility of the geosynthetic to get damaged in the process of movements, then after placing the geosynthetic layer the embankment fill is constructed to required height. And then we can finish our road by putting in the road pavement or railway line.

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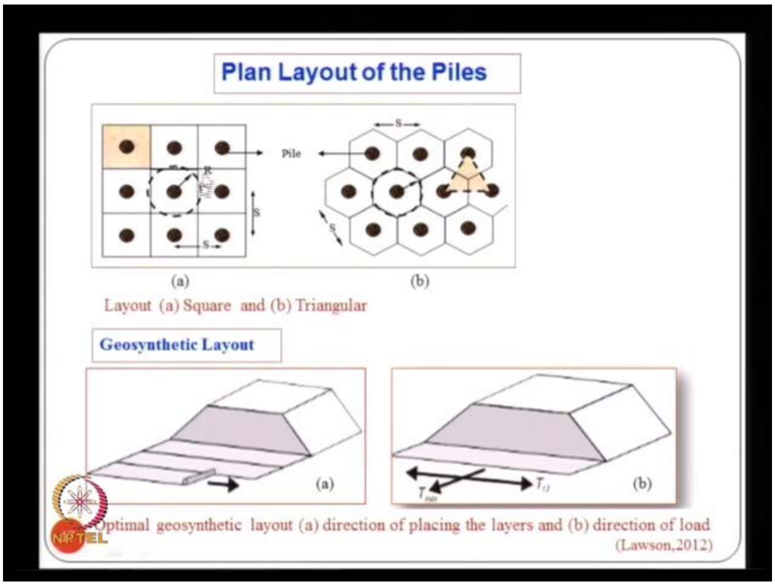

And these are the 2 different plan layouts for the for this piles, either square pattern or a triangular pattern, that we have seen earlier with the case of the P V D's and then the geosynthetic is laid like this, the predominant load direction in the width direction. So, we lay the geogrids like this and the in the length direction, there is either overlap given or some type of seeming is done and the predominant load direction is in the width direction and along the length direction, at the requirement for the load capacities much lower.

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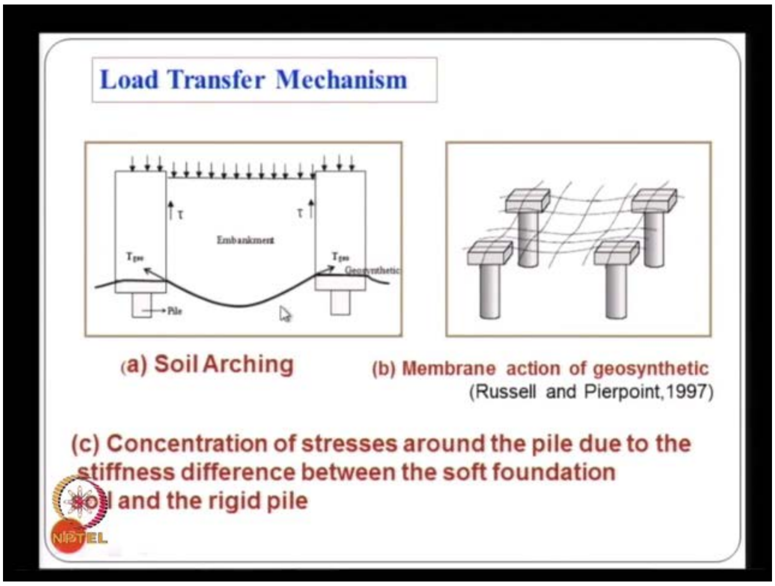

And the load transfer mechanism is very simple, in the absence of any reinforcement all the load is transferred directly into the foundation soil, but because of some friction that is there, because of our inclusions the form of piles and other things. The soil arching may take place that is the concentration of soil pressure the surcharge pressure at some locations where, there is no vertical moment and lesser deformation and lesser pressures are some other places.

Because of the shear strengths, that are developed within the soil body and terzaghi, in his classical papers, he has described soil arching phenomenon and how it reduces the at the pressure transferred on to the subsoil. And because of the placement of the reinforcement, it acts more like a membrane and it assist in further development of the of the soil arching. So, that higher loads are transferred into the piles, so the concentration of the stresses around the piles due to the stiffness difference between the soft soil foundation and relatively rigid piles.

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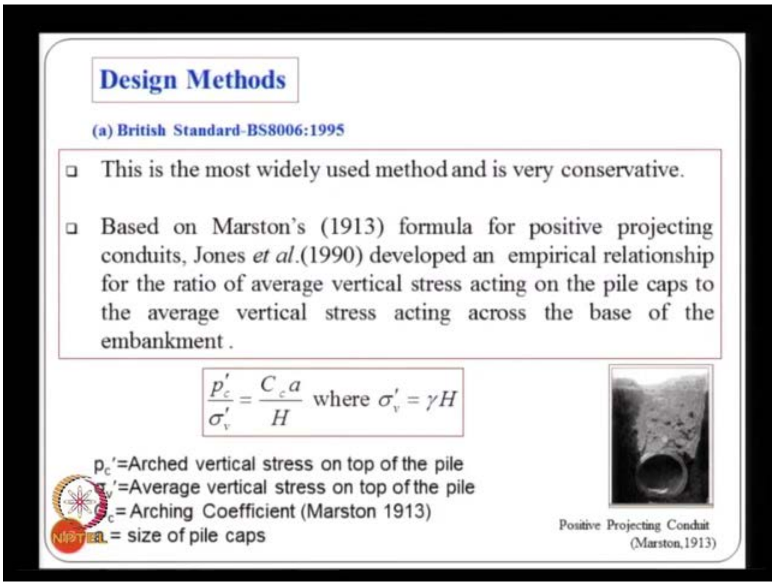

And all the design methods, they relay on the soil arching principles and the one of the most popular design codes is the B S 8006, that is the british standard code, that we have seen earlier and this is the most widely used. And it is also found to be the most conservative among all the different design theories. Jones in 1990, he used the methodology proposed by Marston for the case of positive projecting conduits for developing an empirical relation to calculate the ratio of the average vertical stress acting on the pile caps to the average vertical stress acting across the base of the embankment.

Is actually, the positive projecting conduit is like this, see whenever we have a deep ditch and we have a conduit or a pipe line or the pressure here is not equal to the over burden pressure, but its lesser because of the side friction that is acting. And the same thing is extended by jones and he came out with a methodology to estimate the vertical stress on the piles like this p c prime by sigma v, where sigma v is the average vertical stress on top of the pile, that is the gamma times the height of the embankment.

And that is equal to C c times small a divided by the capital H, here our sigma v is nothing but gamma H, that is the average vertical stress and p c is the is the arched vertical stress, that is much higher than the average vertical stress, because of the soil arching phenomenon and C c is given by marston way back in 1913. And this is called as the arching coefficient and a is the size of the pile caps.

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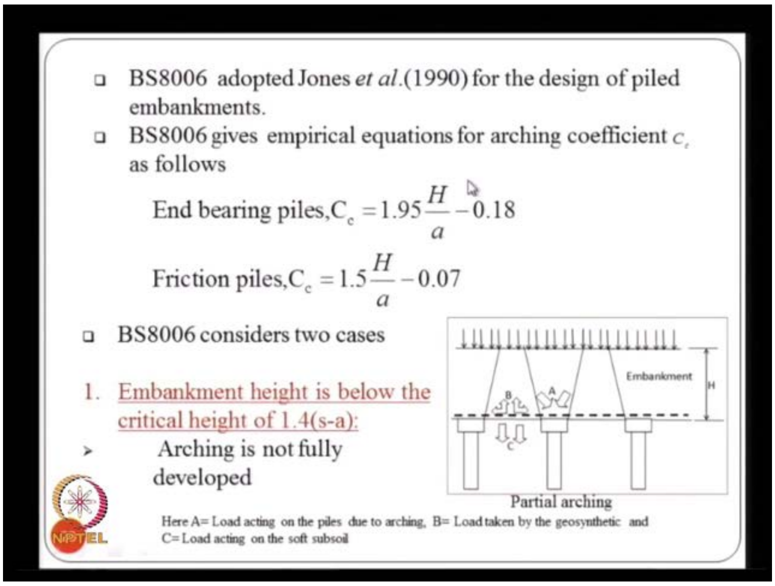

And B S 8006 code, it has adopted the proposition by jones for the design of piled embankments and they have given 2 recommendations for the for this C c, that is the arching factor for 2 cases, one is the end bearing pile. Where the piles transfer the load into the arch stratum, where in there is a large differential settlement between the pile and the soft clay, because the pile itself is does not move. And the other is the frictional pile or what we call as the floating piles, where the pile is not long enough to transfer the load into the deep stratum, it is it floats inside the body of the soft clay.

And it moves along with the along with the soil and in the case of end bearing piles the C c the arching coefficient is written as 1.95 H by a minus 0.18 whereas, for the friction piles the arching coefficient is slightly lesser at 1.5 H by a minus 0.07. And the B S code they have considered 2 separate cases, one is shallow height embankments and the other is very high embankments.

And the shallow height embankments are defined as the embankments, whose height is less than the critical height of 1.4 times S minus a where, S is the centre to centre spacing between the piles and the small a is the pile cap dimension. And so this if the embankment height is less than this it is called as a shallow embankment and we the arching is not fully developed in such case.

And here the different terms shown here or capital A is the load acting on the piles, because of the arching and b is the load taken by the geosynthetic and c is the load acting on the subsoil. And the object of our design is we use a certain spacing of the piles and the geosynthetic stiffness and the strength, such that the load that is transferred into the into the subsoil that is the C component is low as possible this is the C and we want to increase or maximize this capital A and minimize the C.

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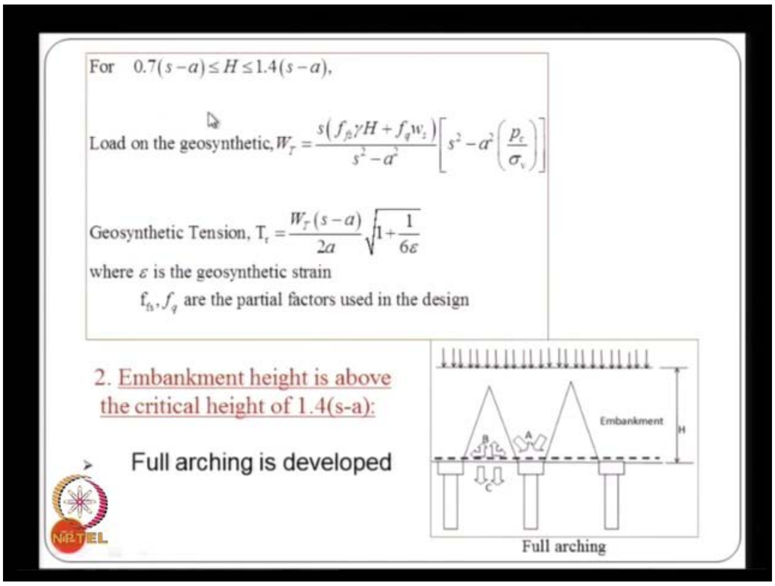

And for shallow height embankments the pressure that is transferred to the geosynthetic is written like this, s times some factor partial factor times gamma H plus some partial factor times surcharge gamma H is the self weight load and the W z is the is the surcharge load. And these are the 2 partial factors this divided by s square minus a square multiplied by s square minus a square p c by sigma v, where sigma v is the vertical stress that is nothing but gamma H plus W.

And the geosynthetic tension is written as in terms of the pressure that is acting, times S minus a by 2 a square root of 1 plus 1 by 6 epsilon, where epsilon is the geosynthetic strain. And the in the design, we have to estimate what is the likely strain level in the geosynthetic. So, that we can calculate the tension and that we will see through some examples later on and if the embankment height is more than this critical height of 1.4 times s minus a, we can assume that full arching is developed.

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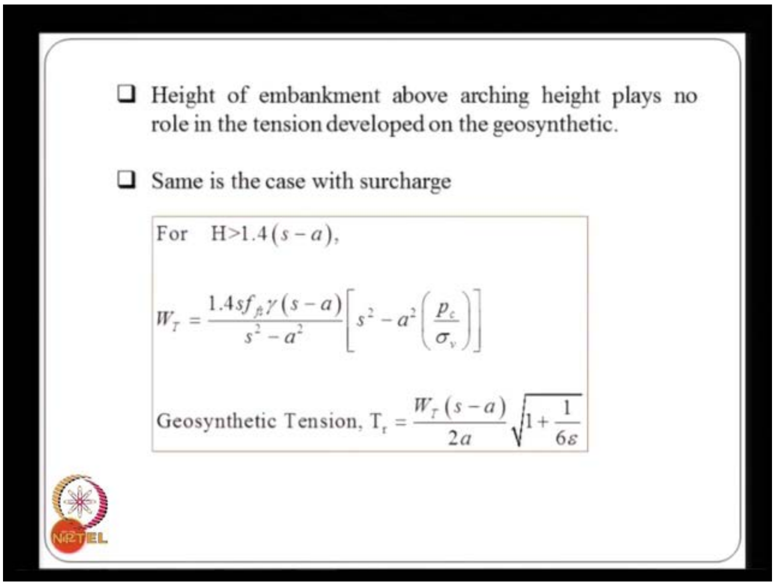

And if the full arching is developed, it is assume the height of the embankment above the critical height does not increase the geosynthetic tensions all that additional load is directly transferred into the piles, our load on the geosynthetic is a given by this. And the geosynthetic tension is given like this.

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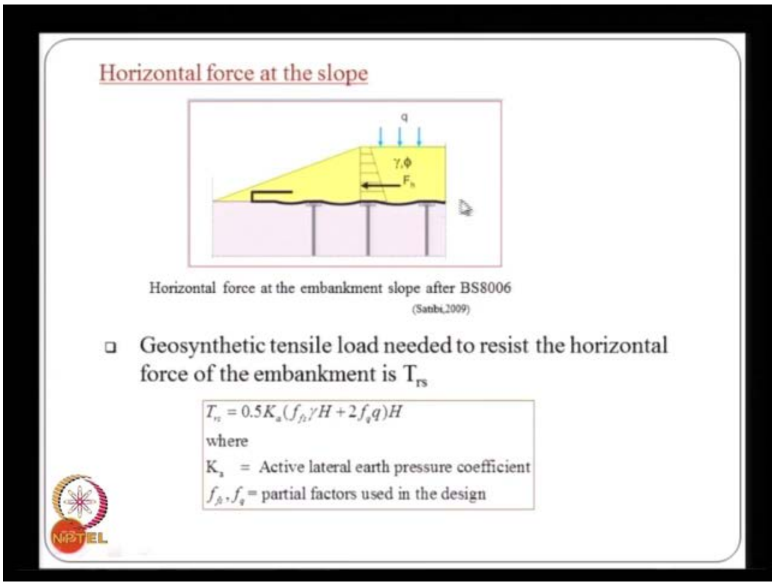

And what are the other factors that, we need to consider especially, when we are dealing with high embankments, we should see the possibility of lateral slip, that is the because we are providing relatively rigid base here, the entire soil may want to flow out by taking this as a weak plane. And this may give rise to additional forces within our reinforcement and to prevent this, we normally provide an anchor block here, that stucks the flow of a the soil.

And the additional geosynthetic force, because of this type of mechanism is 1 half K a gamma H plus the 2 times the q times h. And where our K a is the active earth pressure coefficient, that is applicable for slopes not for retaining walls and our factors are the partial factors used in the design, because we have different of partial factors for load and sorry, the self weight load and the live loads.

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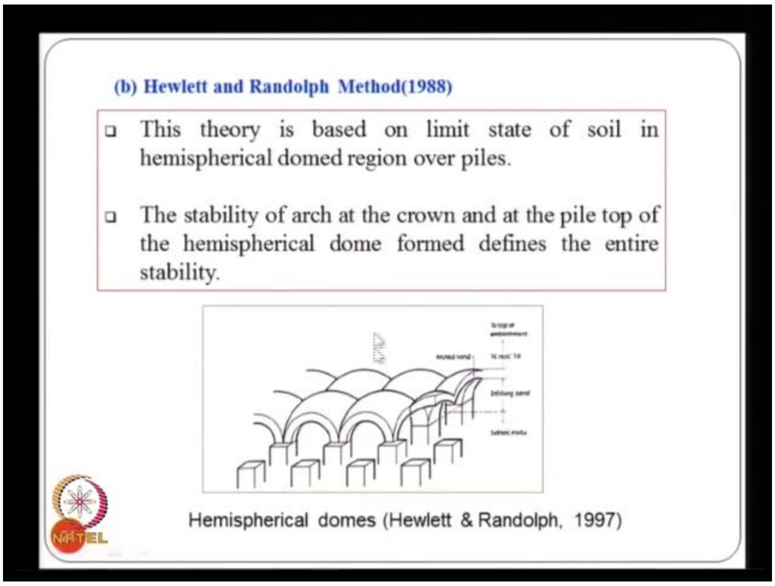

The same arching theory was modified by hewlett and randolph and they considered a spherical type of arched surface, where as terazaghi is considered planar surfaces, they have considered spherical surface. And the they have come out with some other theory to estimate the limit state of soil and then the pressures. And this is how there assumed surface is the deformed as soil surface

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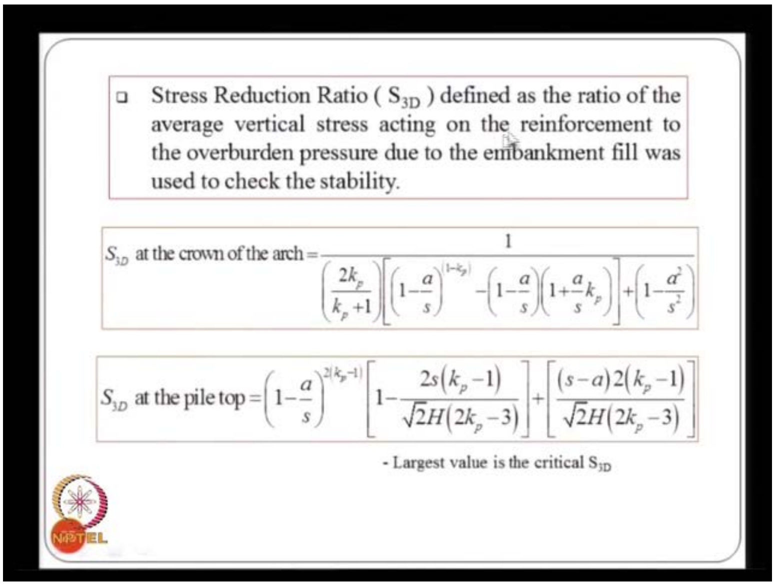

And the stress reduction ratio is defined as the ratio of the average vertical stress acting on the reinforcement to the overburden pressure. And they have developed some equations for the stress that at the crown of the arch and then the pressure, that is transferred into the piles and this is our including the stress intensity factor.

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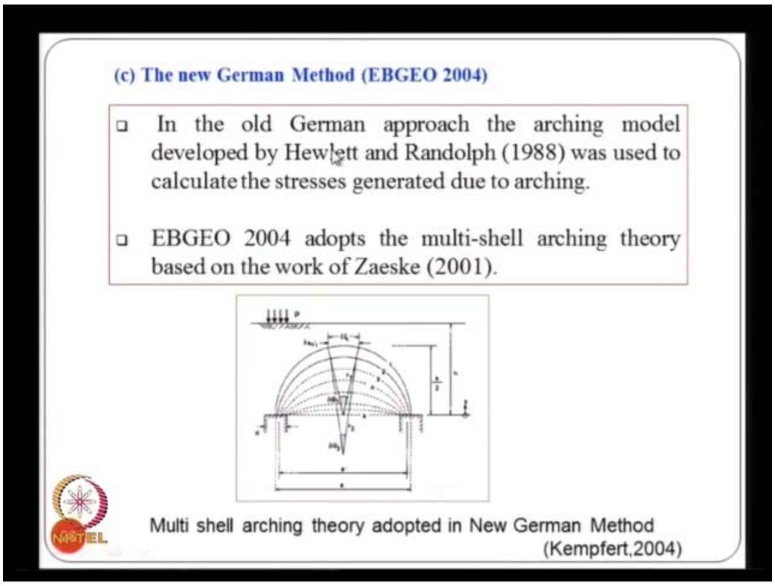

And the german design method is one of the more recent once with, which is bit more realistic. And they employ the original german approach employed the hewlett and randolph theory to calculate the stresses generated, because of the arching on the geosynthetic and also the stresses on the that are transferred into the pile foundations. And the more recent version 2004, it adopts slightly different approach, that was proposed in 2001 and the once again this is also this theory assumes some spherical shell type surface for estimating the pressures.

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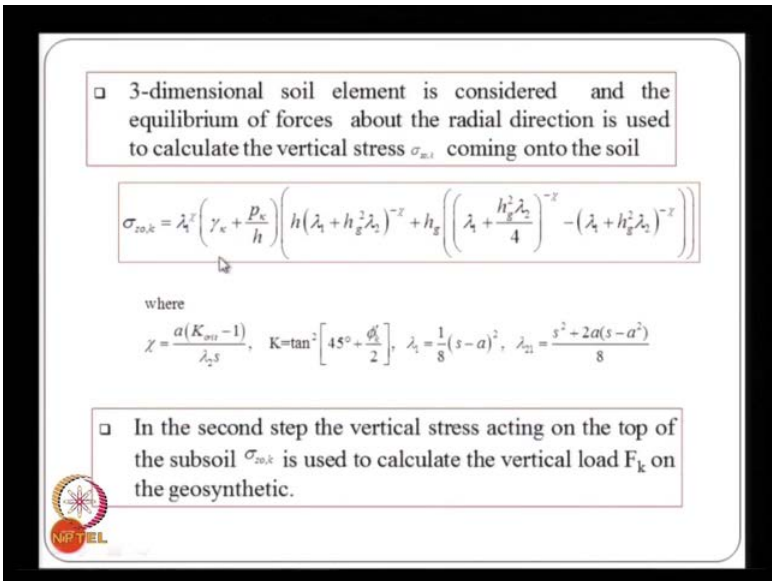

And these are the pressures, that are calculated based on this based on this approach, basically the pressure that is transferred on to the geosynthetic and the soil their functions of the unit weight. And then the applied loading and all other factors like the factors related to the spacing between the piles and so on.

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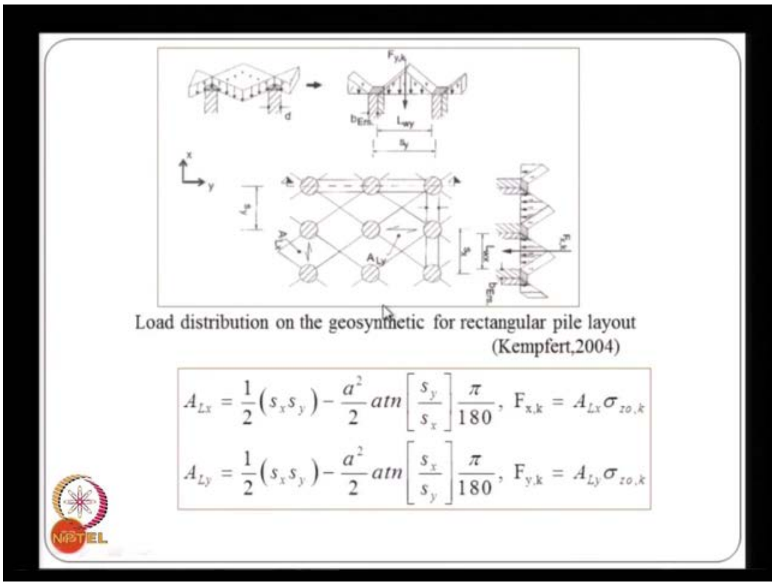

And these are some of the mechanisms, that were outlined by Kempfert, where we can calculate the force, that is transferred into the piles through these equations.

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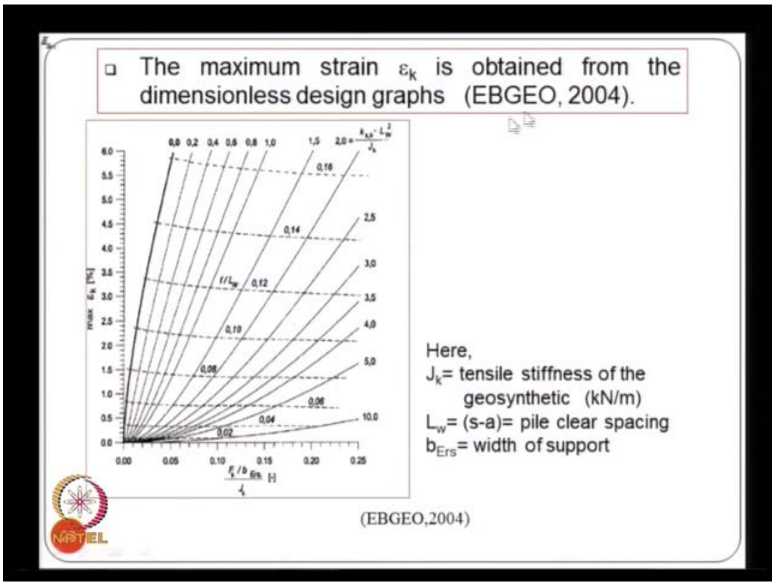

And the this is once again from the German code, the more recent one version the maximum strain developed in the geosynthetic reinforcement is expressed in terms of some factor. The f by b where our different factors b is the width of the support, that is the pile cap width and l is the spacing the clear spacing between the piles s minus a, s is the centre to centre spacing and a is the pile cap width. And j is the tensile stiffness and we can get the normalized force in the geosynthetic, in terms of the strain and then other arching factors.

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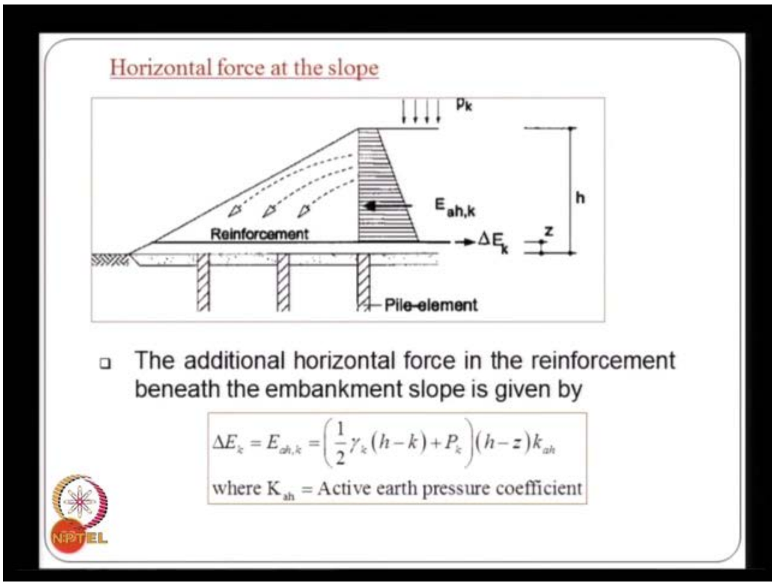

And once again, even the german code, they also recommend that additional force, that is generated in the reinforcement, because of this lateral thrust is considered.

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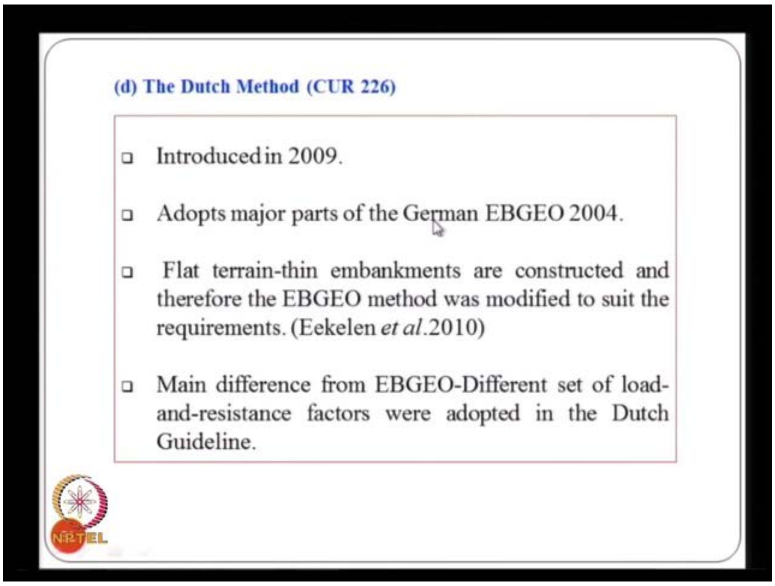

And more recently the Netherlands in 2009, they have introduced their own design methods, which is modified form of the German code. And is actually this the in Netherlands the height of the embankments are or much fatter. And then the they have adopted the German code to suit their own geometry. And the main difference between the German code and the Dutch code is in the load and resistance factors, there is some difference in this factors.

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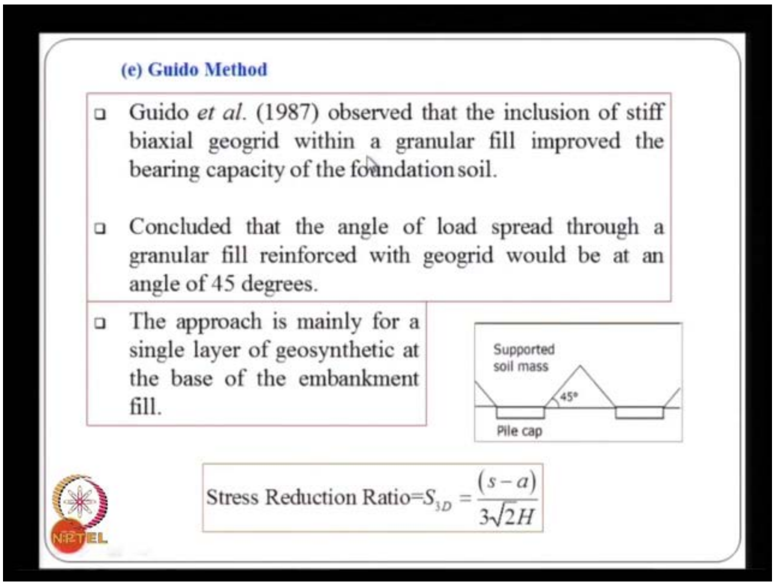

And guido et al in 1987, they also proposed some theory, wherein they assumed that instead of domed surface, we can assume this arched portion as a triangular portion, which is at 45 degrees, they observed that the inclusion of stiff biaxial geogrid improve the bearing capacity of the foundation soil. And the we can assume that, the load is filled through the granular fill at an angle of 45 degrees, that is a simplified form of the assumptions.

And the pressure, that is transferred into the soil is only, because of this triangular portion where as, the load that is transferred into the piles through the pile cap is because of all this area. And so if we are able to minimize the spacing between the piles, we have higher loads on the piles and the stress reduction ratio is written as s minus a by 3 times square root 2 times H, that is the reduction factor, for this stresses within the soil.

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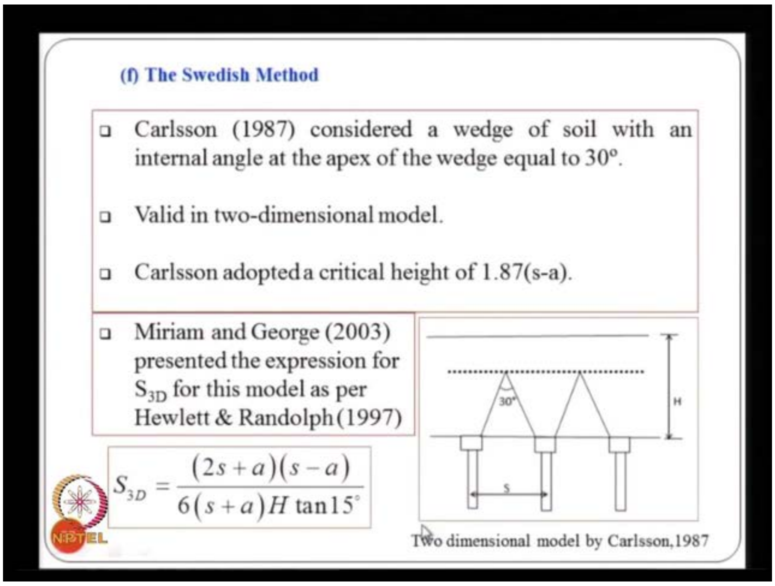

And there is another Swedish design method, wherein our the angle is only assumed as a 30 degrees not as 45 degrees as in the earlier case or this angle is 60 degrees, where as in the guido's procedure, it is 45 degrees. And in the Hewlett and Randolph it is a spherical type dome surface and the Swedish method, that was proposed by Carlsson in 1987 considers the wedge of soil within an internal angle at an apex angle of 30 degrees. And this is valid in the 2-dimensional case and he adopted a critical height of 1.87 times s minus a whereas, the B S code, they have consider on1.4 times s minus a. And their, stress concentration factor is given like this.

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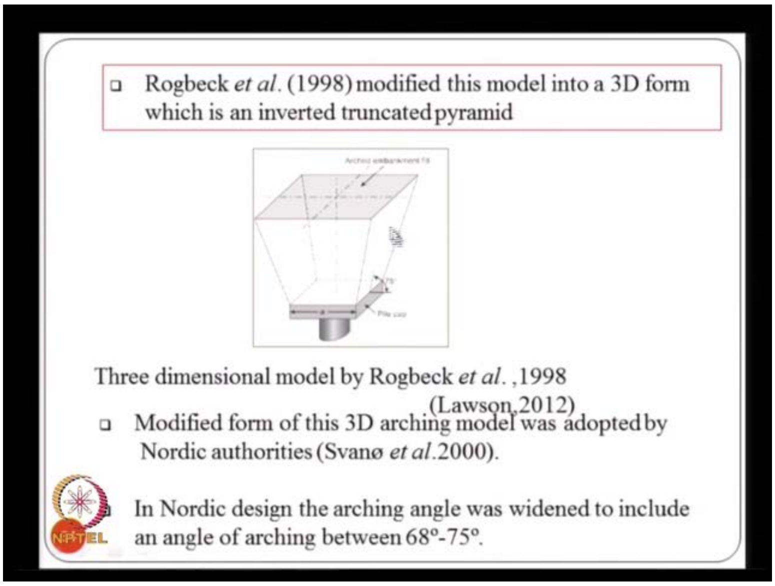

And another form of looking at it is by Rogbeck, wherein in Pyramidical shape of soil considered, this is the case of 3 dimensional analysis.

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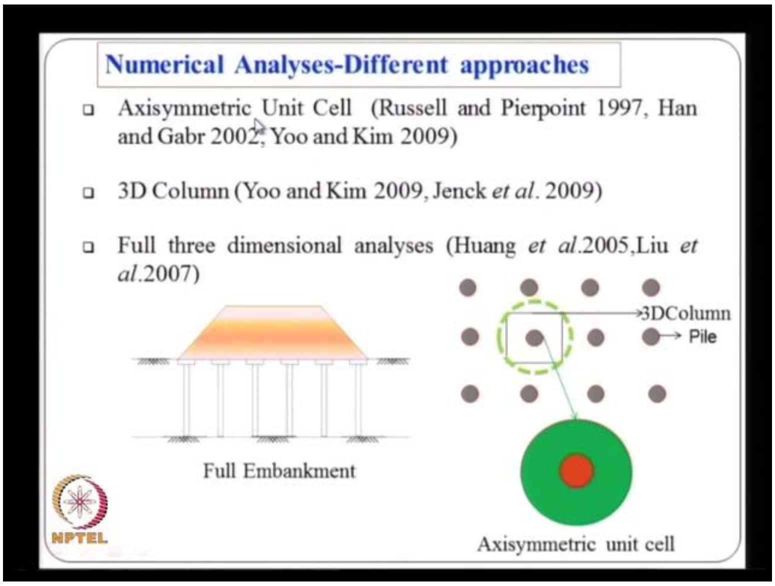

And very large numbers of finite element analysis have been done to understand this phenomenon. And these are all some of these approaches and we can have an axisymmetric unit cell wherein, we consider some part of the soil and one of the single piles as a part of a unit cell and perform the analysis for circular geometry.

And the same thing can be done in the 3 dimensional form by considering this area as a 3 d column and the ultimate one is performing full 3 dimensional analysis finite element analysis. In terms of the expense, the last one is the most comprehensive one, it which may give the best of the results, but then it takes lot of computational effort. Because, the size of the module is very large whereas, the first 2, they are equivalent and they are much smaller and they are very fast to run.

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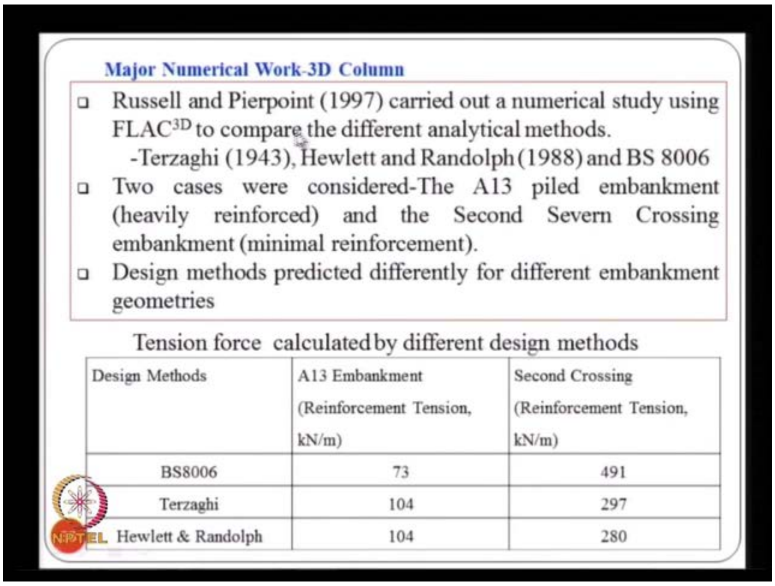

In some of the early numerical analysis were performed by russel and pierpoint, using the flac program, that is based on the finite difference approach. And they compared the results by different approaches like the terzaghi hewlett randolph and then the B S code and they considered 2 geometries one for the a 13 piled embankment, which is relatively heavily reinforced and the second 7 crossing, that has a minimum reinforcement. As we have seen in the second 7 crossing, there are only 2 layers of biaxial geogrids, each having a tensile strength of 20 kilo Newtons per meter.

And the different design methods, they have shown different results, for example, for the a 13 embankment the B S code, they have predicted only 73 kilo Newton per meter force whereas, terzaghi and the hewlett randolph approach, they have predicted the force of 1 naught 4. Whereas, in the second seven crossing, where the reinforcement is minimal and the spacing between the piles is relatively large. The B S code, thay have predicated very high force of 491 whereas, terzaghi and hewlett Randolph, they predicated similar forces, but much smaller 297 and 280 almost half of the force predicated by B S code.

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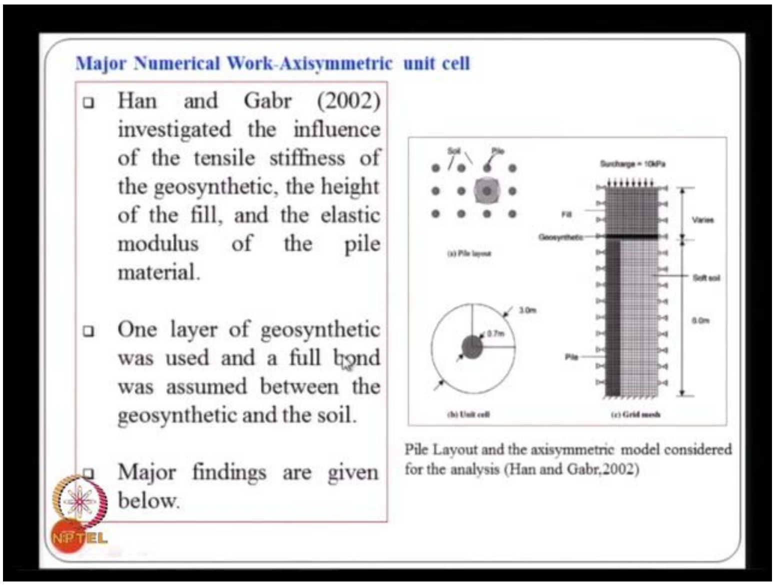

Some of the numerical approaches are like this the axisymmetric approach that is very simple to perform and also very popular, because it takes lesser computational effort. And Han and Gaber in 2002, they have investigated the effect of the column stiffness of this piled embankment and then the tensile stiffness of the geosynthetic and they came out with some recommendations. And their numerical module is like this, so when you have pike arrangement like this, we consider a single pile and some soil around the pile, that is influenced by each of this pile.

And we develop a finite element module like this and this dark area is the pile and this horizontal line, thick line is the geosynthetic and above this is the reinforcement sorry, the reinforced the soil fill. And the below the geosynthetic reinforcement is the soft soil and they have performed several analysis by changing the stiffness of these of the pile. And then the stiffness of the geosynthetic on the strength and they have considered only 1 layer of geosynthetic and full bonding is assumed, they did not allow for any slip between the geosynthetic and the soil.

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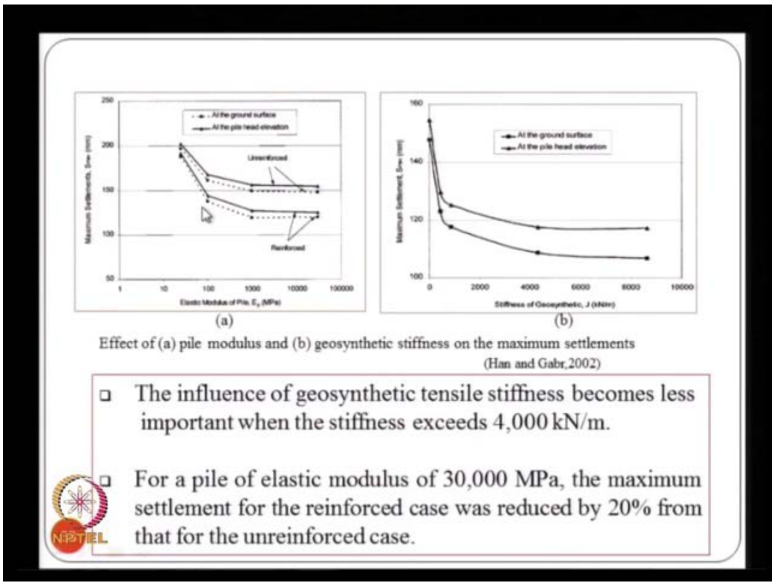

And some of their findings are like this and they have shown the relation between the elastic module of pile and then the maximum settlement, the foor unreinforced case and for the reinforced case and with reinforced settlements are found to be lower. And the settlements shown with the dashed line, these are at the ground surface, that is at the top of the embankment whereas, the solid lines, they show the settlements at the pile head or the foundation soil level.

And we see that as the elastic modulus of the pile increases the settlement reduce, but beyond certain value about 1000 N P A, the settlements are more or less constant, that means that, even if you increase the pile modulus the settlements are not going to reduce. And on the right hand side, the result by varying the stiffness of the geosynthetic, they varied up to almost 8000 kilo Newton per meter and we see that after about 4000 kilo Newton per meter stiffness.

These lines are flat; that means, that further increase in the geosynthetic stiffness at the settlements are not going to reduce. So, their recommendation is the influence of the geosynthetic tensile stiffness, becomes less important. When the stiffness exceeds four thousand kilo newton per meter or when the elastic modulus of the pile material is more than about 3000 N P A, the maximum settlement is not very much influenced where, there you have 30000 or 40000, it is not going to be very different.

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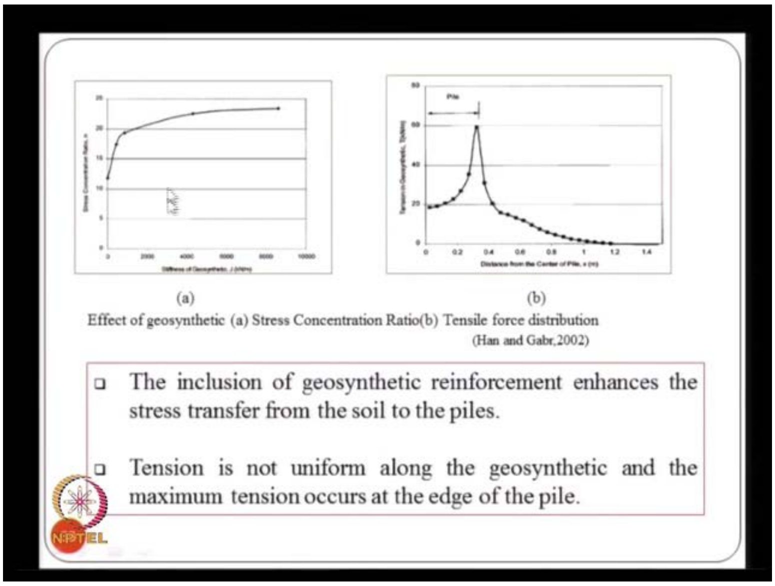

And the stress concentration factors, they have given against the stiffness of the geosynthetic, basically the stress concentration factor is the stress on the pile divided by the average vertical stress that is the gamma H. And when we do not have any reinforcement the stress concentration is about 11 or 12, but as we increase the stiffness of the geosynthetic, the stress concentration factor increases at increases almost about 23. And that means, that as we increase the stiffness of the geosynthetics, we can transfer higher loads into the into the piles.

And once, we transfer the higher loads into the piles the settlements, because of the transfer of stresses into the foundation soil or lesser and the tension force developed in the reinforcement is not uniform. And the maximum tensile force happens at the edge of the pile ass shown here, because that is, where we have the maximum relative deformation between the pile and the adjoining soil. And then the away from this the reinforcement force reduces.

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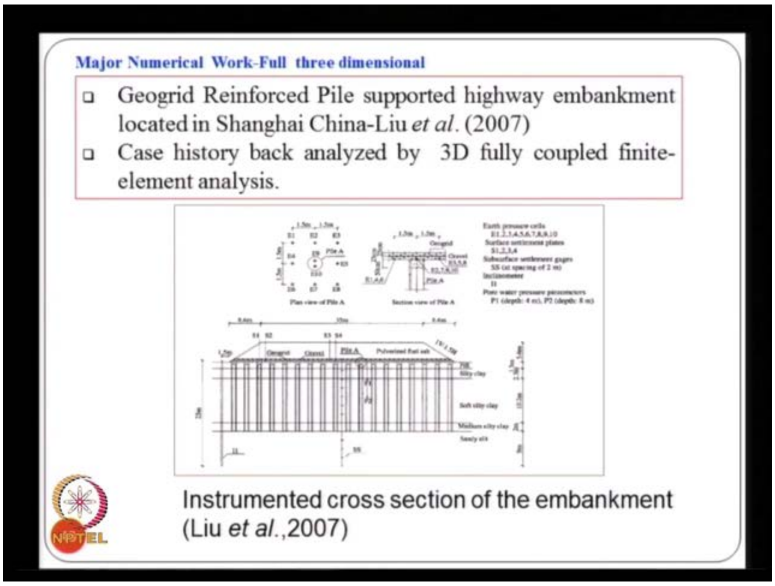

One of the first full 3 dimensional analysis was performed by Liu and others. And they have numerically analyzed, the published data on a geogrid imposed pile embankment that was constructed in shanghai and they have performed the full 3 dimensional analysis

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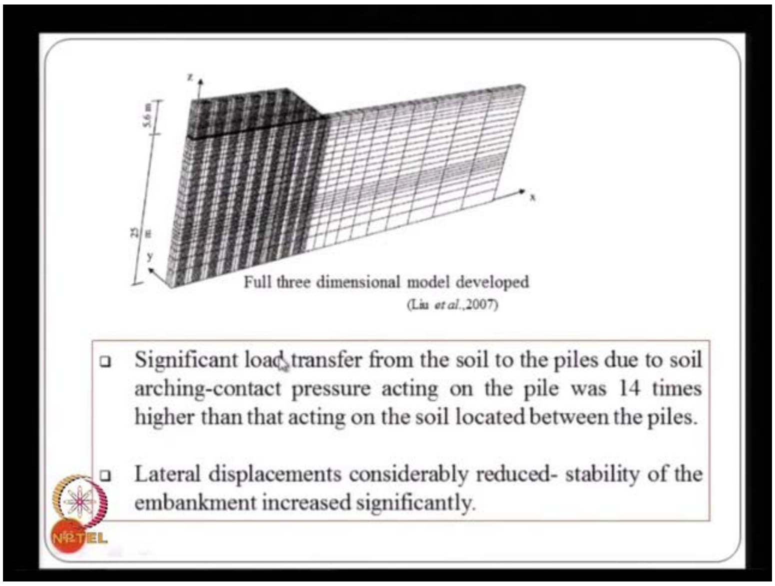

By considering the solid elements and they found, that the significant load transfer happens from the soil to the piles, because in soil arching. And their contact pressures predicted from full 3 dimensional analysis are about 40 times higher than the tacting on the soil located between the piles.

And the lateral displacement considerably used, because of the placement of the piles and the stiff reinforcement and because of that, the stability of the embankments has increased significantly.

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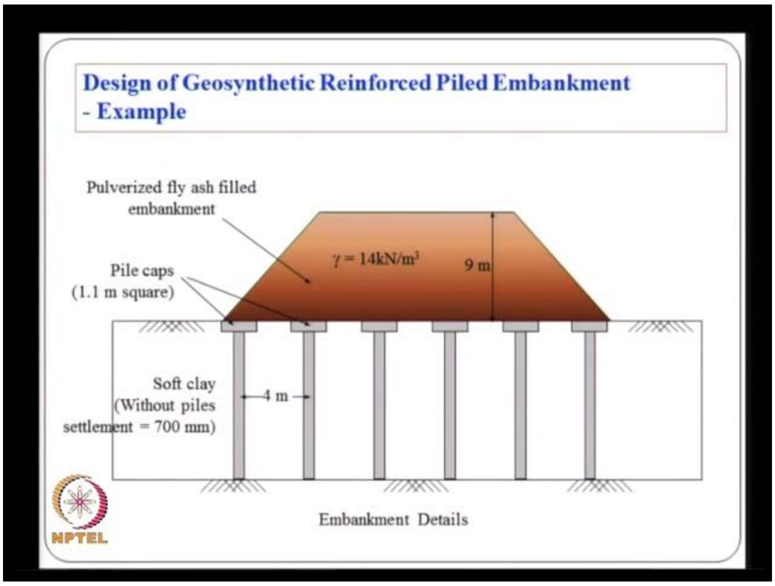

Now, let us consider, simple numerical example on the design of reinforced piled embankments and let us say that, we want to build a 9 meter high embankment, using the pulverized fly ash or pond ash having a unit weight of 14 kilo Newtons per meter. And let us say that our piles that placed at 4 meter centre to centre and with and the settlement without any treatment is estimated at about 700 m m and the pile caps, they are 1.1 meter squares.

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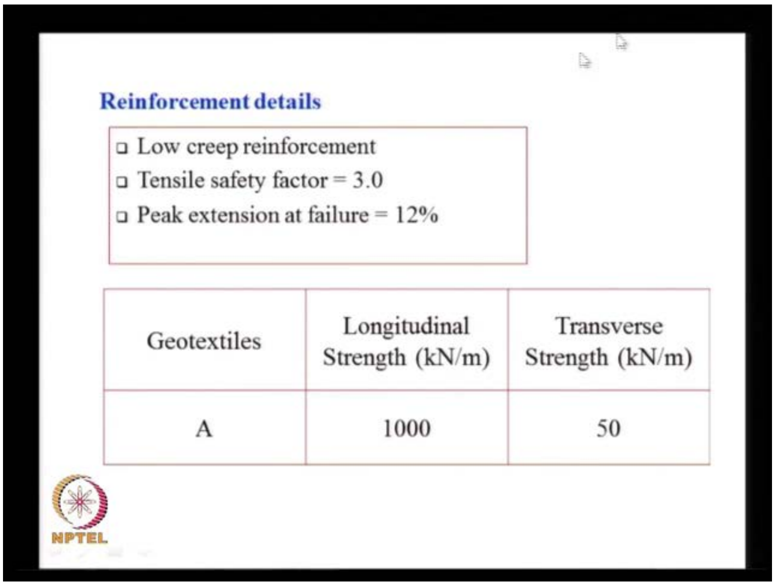

And lets use a tensile load factor of 3 and peak extension at failure is 12 percent and lets use a relatively low creep reinforcement and lets use a geotextile, that has a longitudinal strength of 1000 kilo Newton per meter and strength in the transitional direction of 50 kilo Newtons per meter.

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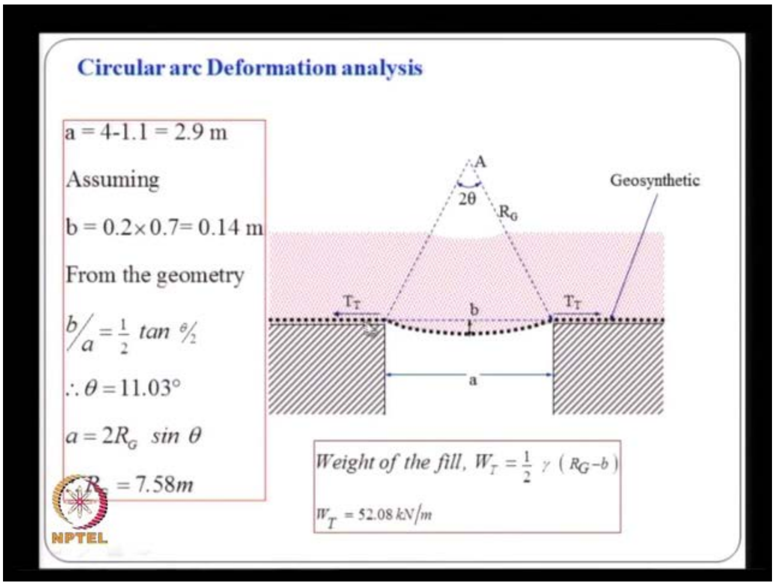

And is actually, we can assume different geometries for this deformed shape and we have assumed that.

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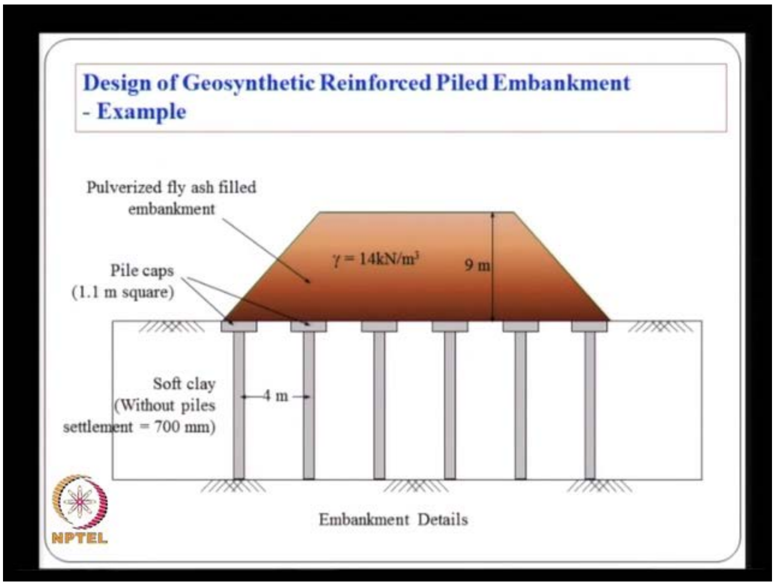

The settlements without any treatment or 700 m m.

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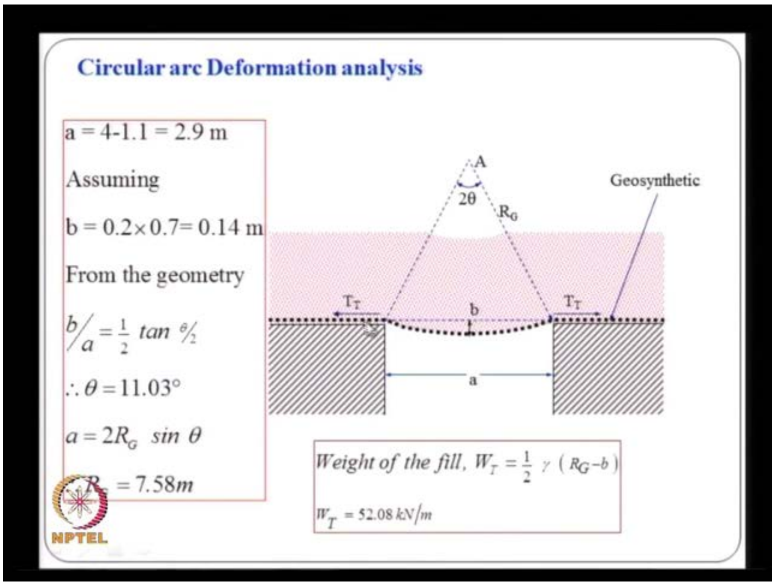

And lets say that, because of the placement of the geosynthetic pile system our settlements are only 20 percent of the settlements of the untreated ground is about the maximum settlement is about 140 millimeters. And if you assume that, between the 2 supports the geosynthetic undergoes a circular arcs type deformation, we can just construct this geometry.

And then calculate that the radius of this imaginary deformed surface is 7.58 meters and the weight of the soil fill, acting on the geosynthetic is 52 kilo Newtons per meter, these are all from the british code.

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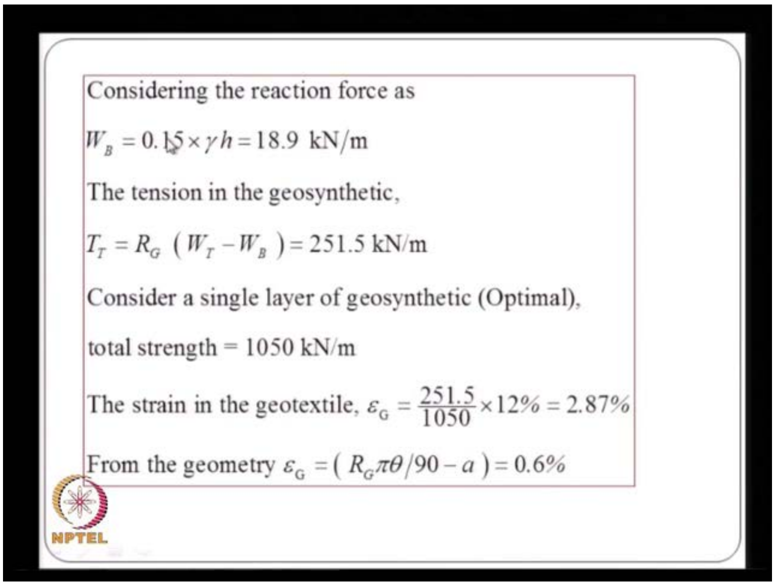

And the considering the reaction force, that is coming from the settlement as 1.15 times gamma h, the tension force developed in the reinforcement is this r g times the vertical minus the reaction force, that is about 250 kilo Newtons per meter. And let us consider a single layer of geosynthetic reinforcement with 2 layers, one is longitudinal and the other is transverse, so the total strength is the total strength provided is 1050 kilo Newton per meter. And the strain within the reinforcement at this load is if we use a linear proportion, because at 12 percent strain, this maximum load is developed, the strain corresponding to this is only 2.87.

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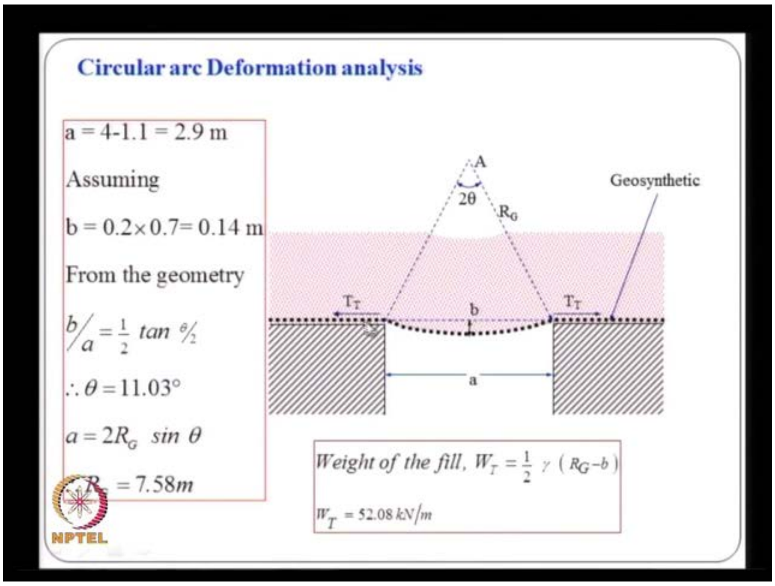

And from the geometry of this deformation, we can calculate the strain in the reinforcement.

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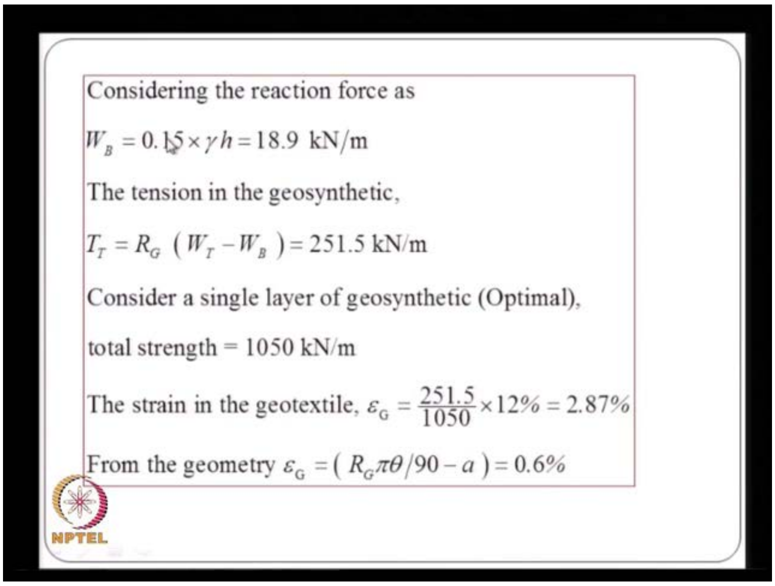

And the strain happens to be only 0.6 percent, so there is a large difference.

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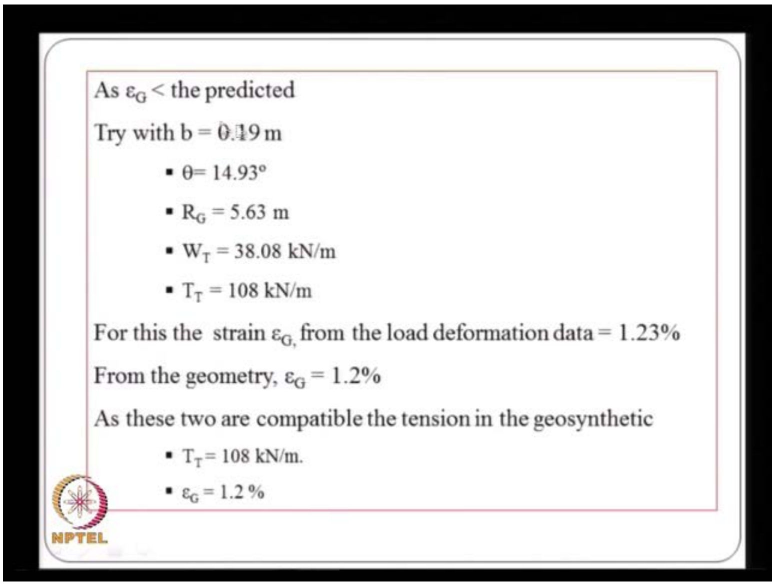

So, we need to revise the calculations and let us assume that, the settlement is 190 millimeters as oppose to 140 millimeters, in the previous case. And so from this, if you redo the calculations the strain in the reinforcement comes as 1.23 percent and from the geometry of that slip circle, it is only it is 1.2 percent, which is very close enough. And so this design of providing reinforcement and as assuming the settlement as 190 millimeters seems to give compatible reason design and so the estimated tensile force in the reinforcement is 108 kilo Newton per meter. And then the strain is 1.2 percent.

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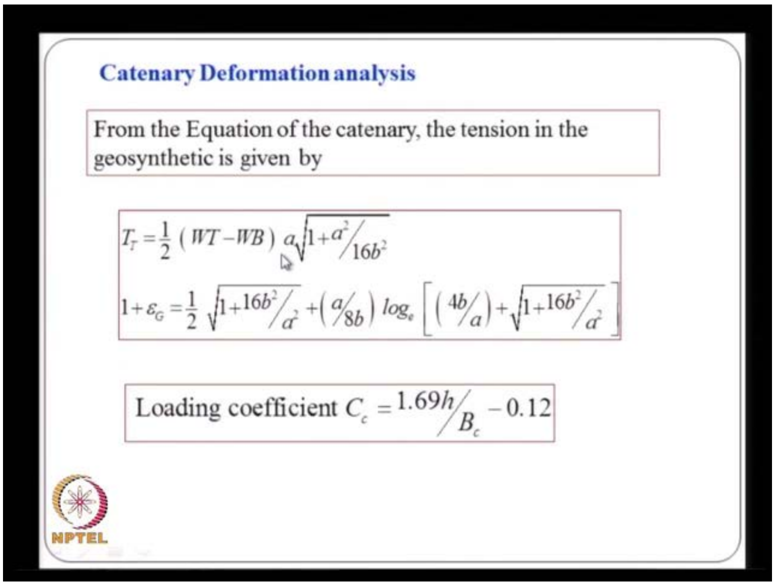

And we can also assume different shapes the catenary type of deformation and we can calculate under this is the archong coefficient.

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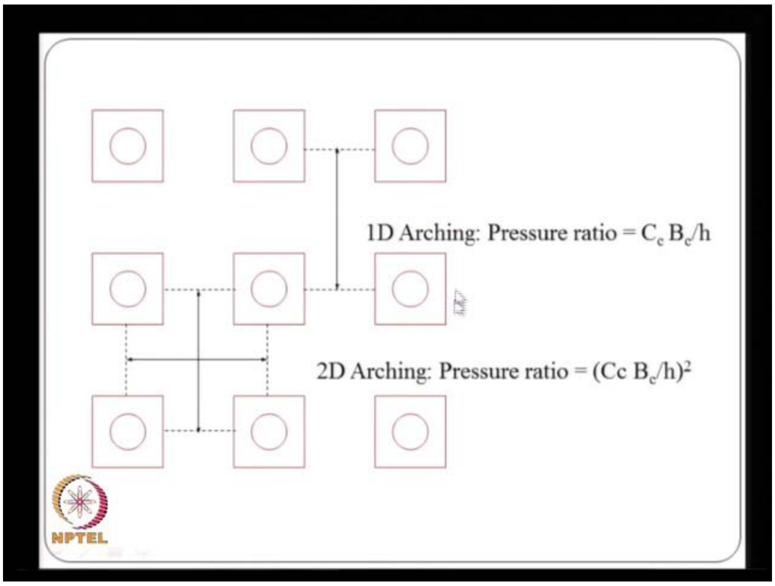

And we can use one dimensional arching theory or 2 dimensional arching theory, the if you use 1 D arching theory, the pressure concentration ratio C c B c by h whereas, in the 2 d arching, this gets squared C c B by h square.

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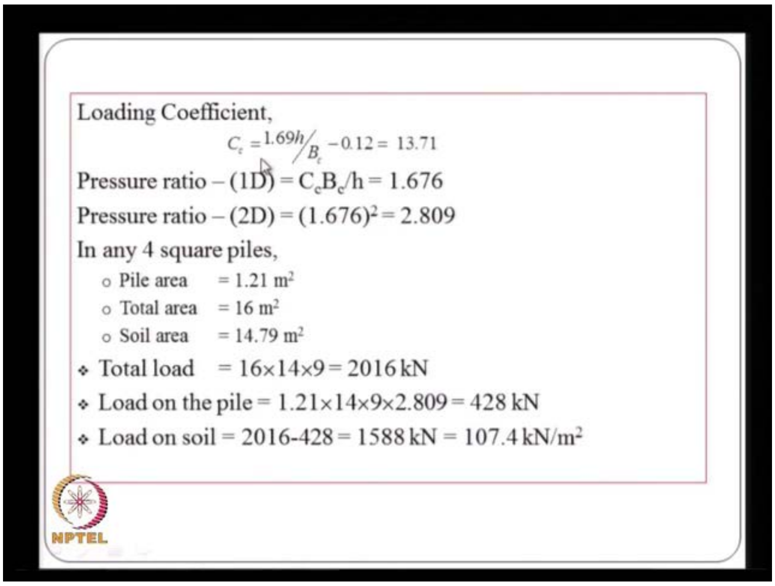

And the loading coefficients are shown like this and for pile area of 1.21, the total load coming from the surface is 14 is the unit weight and 9 is the height of embankment and the plan area, that is supported by between the piles is 16 square meters. So, it is 2016 and the load on the pile is considering the arching factors is 428 kilo Newtons load on each pile and the load on the soil is and the pressure is 107 kilo pascals.

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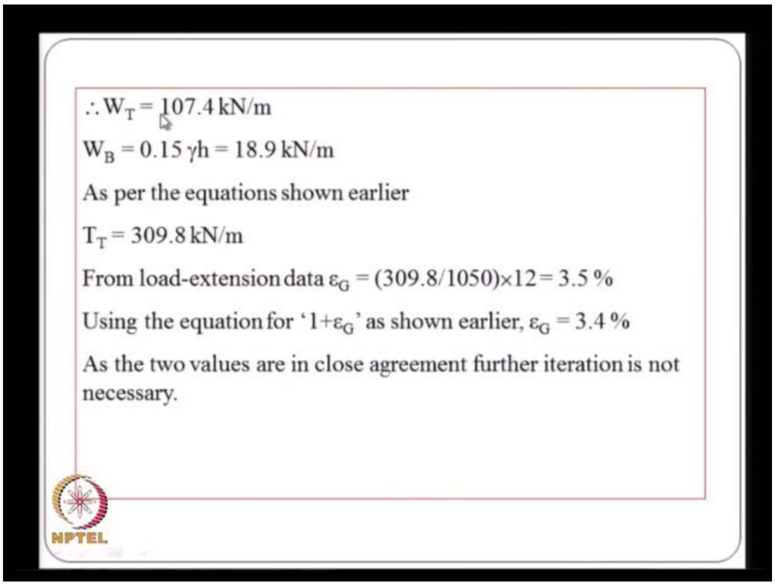

And the weight acting on the reinforcement is 107 kilo Newtons per meter and the tensile force, we can calculate as 309 kilo Newtons per meter using the previous equations. And the from the load extension data, this load of 309 kilo newton per meter corresponds to a strain of 3.5 percent and the geometry of data assumed deformation shows strain of 3.4 percent. So, which are in a very close to each other, so this design is valid.

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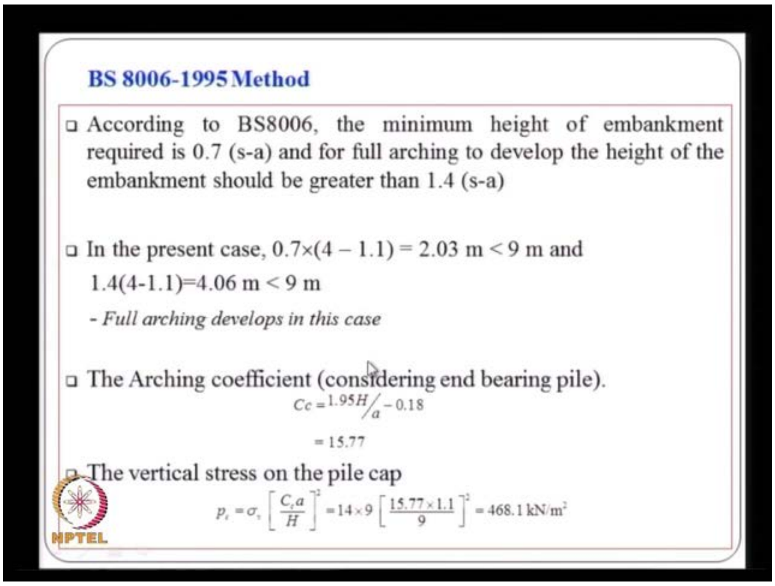

And the same design by using B S code is it is like this, we can and our embankment is 9 meters size, which is more than the critical height, we can estimate that, the arching is fully developed, the arching coefficient for embearing piles is 15.77 by substituting all the values here, then the vertical piles of the pile cap is this.

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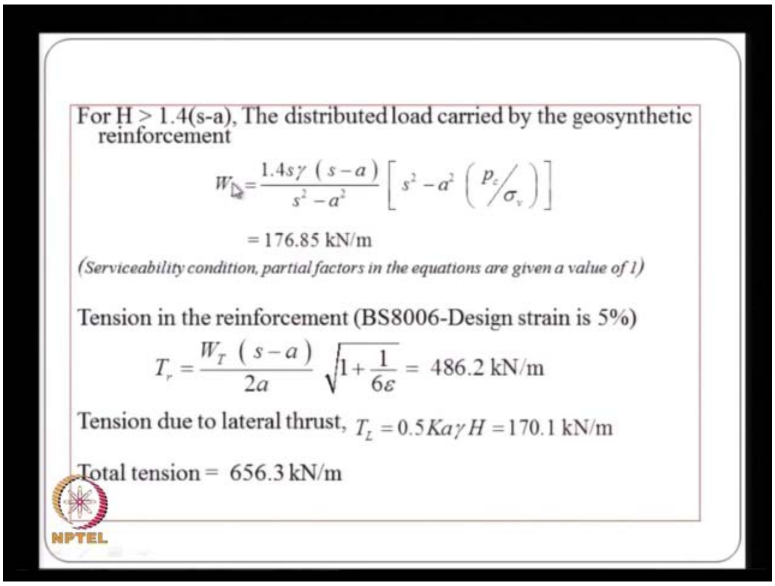

And the distributed load on the geosynthetic and then the tensile force developed.

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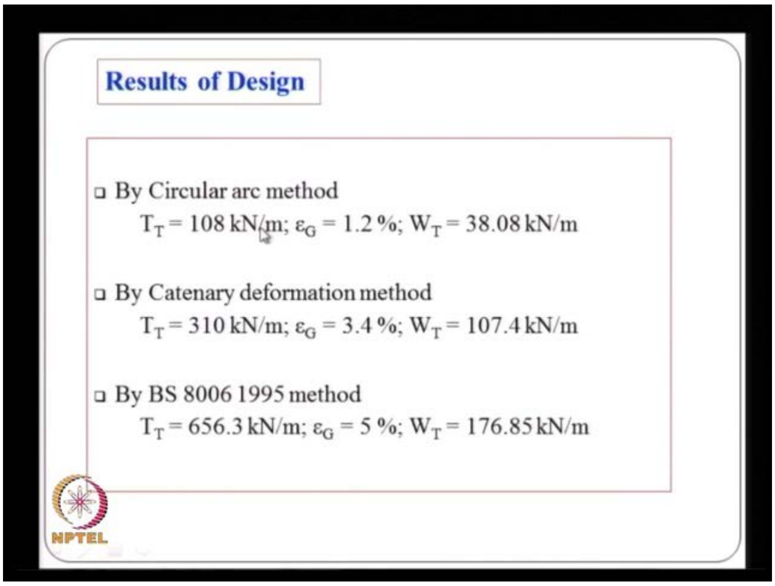

And the comparison in the results between the different methods, where is circular arc method, the tensile force in the reinforcement is 108 kilo Newton per meter whereas, the catenary deformation method. It shows 310 whereas, the B S code, it shows much higher force of 656, which is very high and the level of the stress mobilized are also very different 1.2 percent by the circular arc theory whereas, the other one is almost 5 percent strain.

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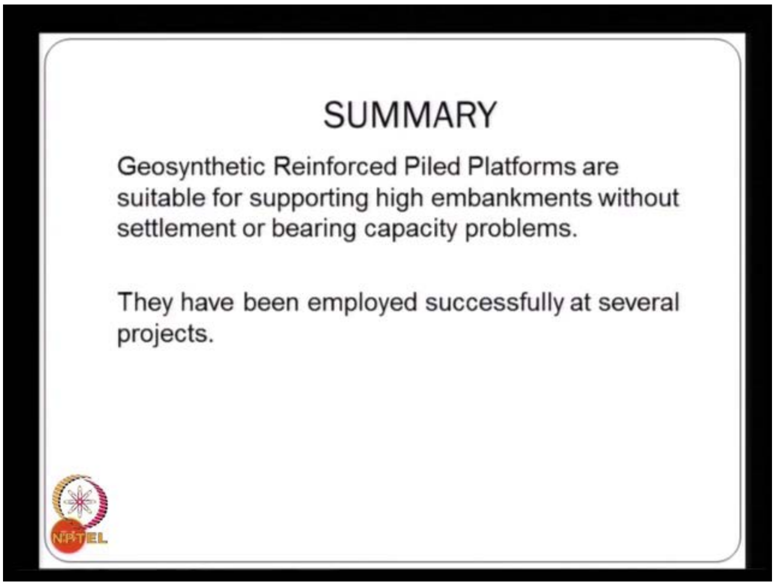

So, just to summarize the geosynthetic pile embankments are successfully applied for all for all the constructions of very high embankments, some resting on soft clay foundation.

Thank you very much.