

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

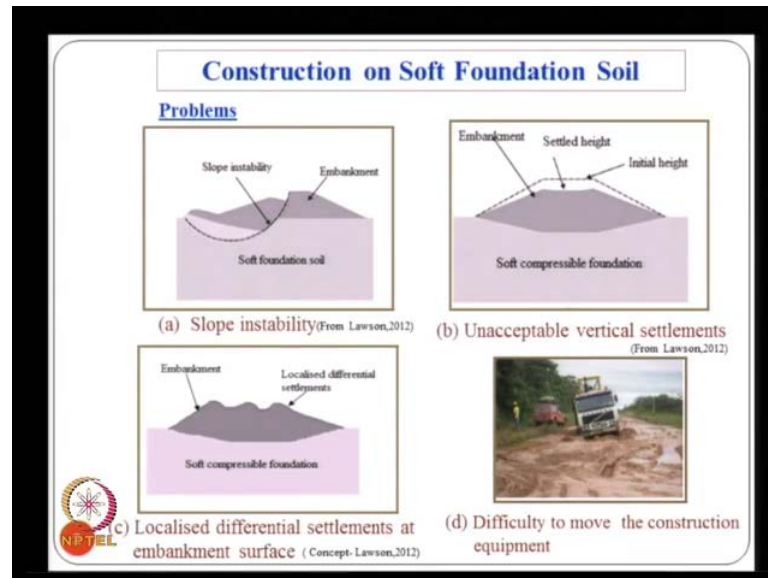
Very good morning students, let us continue our discussion on the construction on extremely soft clays.

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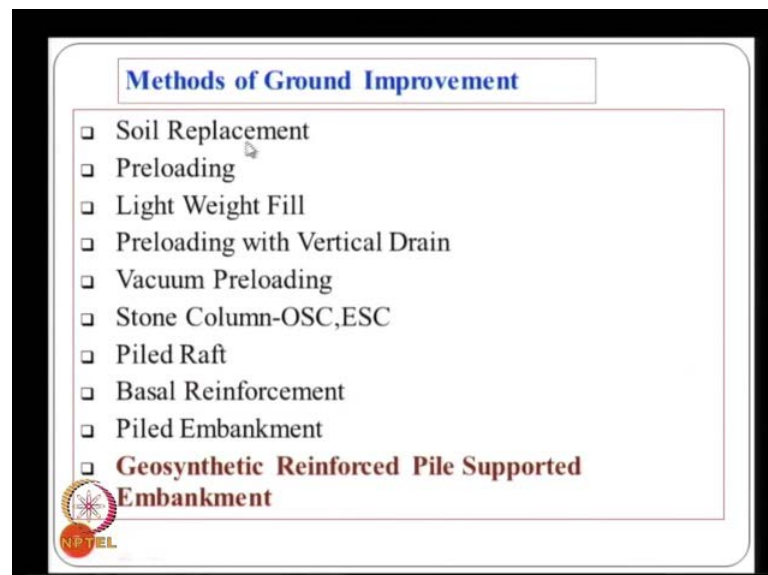
One option is the use of Geosynthetic Reinforced Pile Embankments, and let us see what they are and how we employ them for construction in this lecture.

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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.

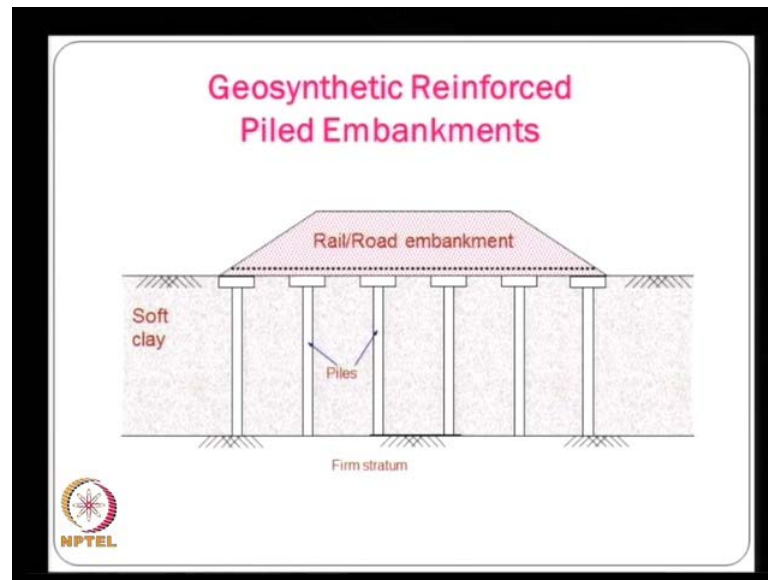
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And we have already seen that, there are several options, that are available for construction on a soft clays, that is the easiest one is the soil replacement, if the thickness of the clay soil is very small. We can think of replacing or the preloading that we have

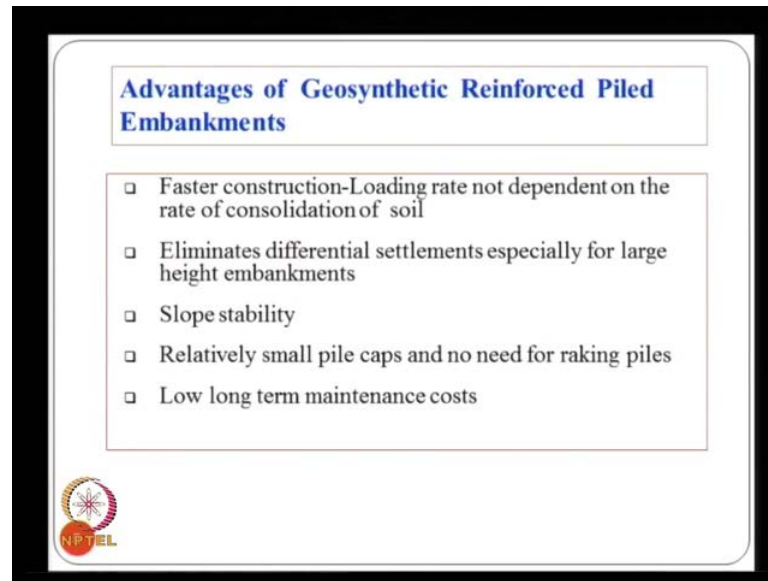
seen in earlier lectures either by surcharge consolidation or by vacuum consolidation, and using the light weight fills like polystyrene or pond ash and so on. And the most recent one of all these techniques is the geosynthetic reinforced pile supported embankments.

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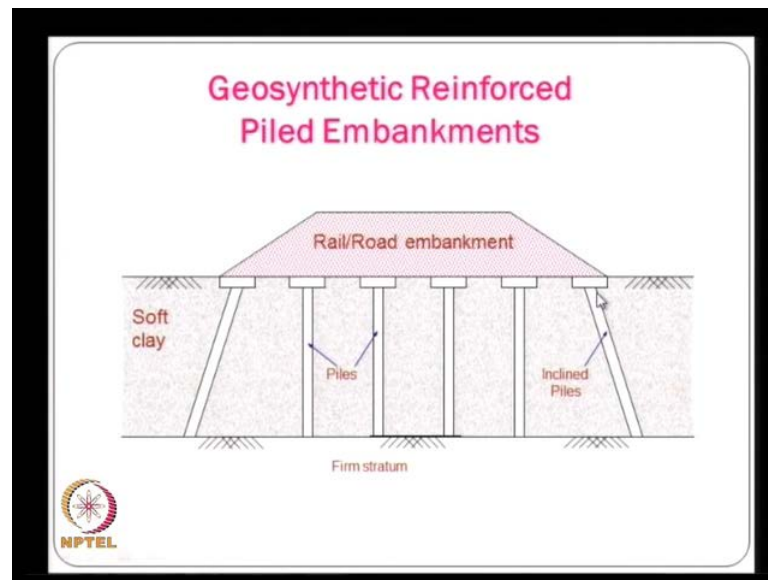
And this has actually become very popular especially for very high embankments, and for very thick clay soil deposits. Conceptually, the geosynthetic piled platforms are like this. And the concept wise it is similar to pile raft, where we have a solid raft reinforced concrete raft and the piles except 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 plain 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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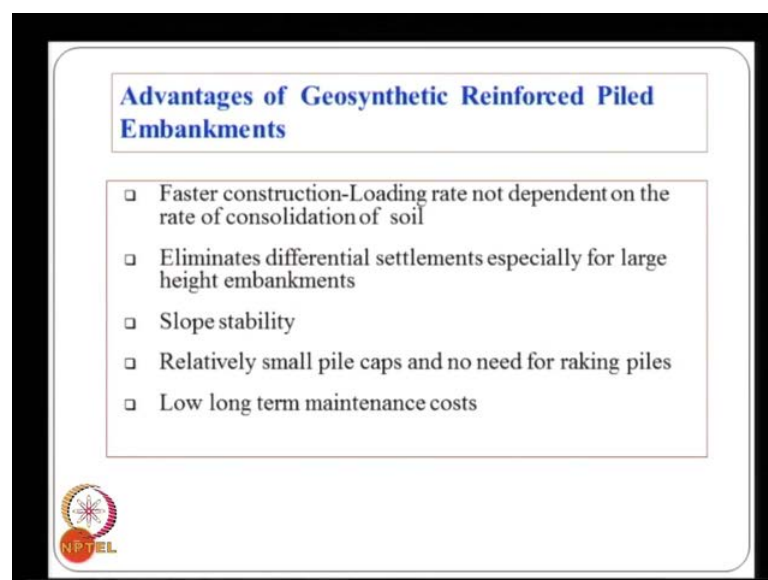
And some of the advantages that, we have with this type of construction is we can have a faster construction and the loading rate is not dependent on the rate of consolidation of soil. Because, now we are not depending on the on the strength of the soil, but rather on the strength of the whole system 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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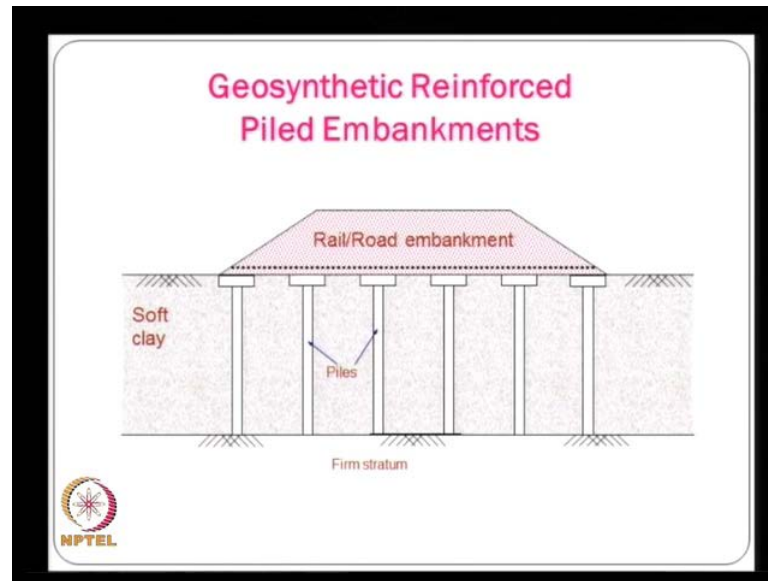
Especially, the pile raking is something at the end of the embankments, we give the inclined piles to take care of the lateral thrust from the high embankments. And because the provision of horizontal of layers of reinforcement all the lateral load is taken by the reinforcement, and because of that the 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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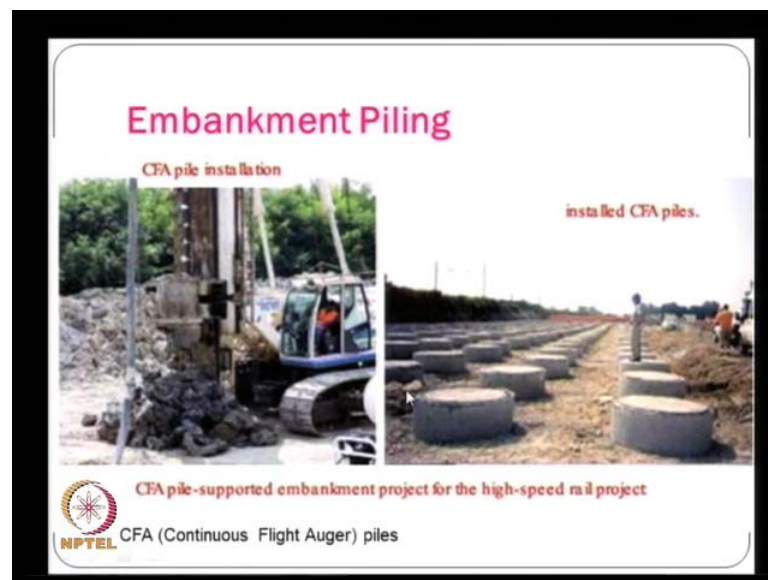
And is also, we will have very low maintenance costs, because the differential settlements on the total settlements, that are expected from the systems are small.

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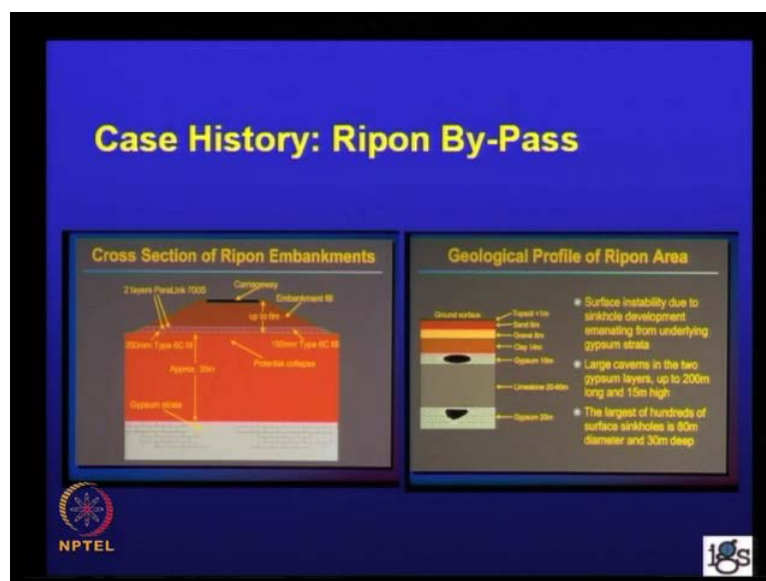
And let us see some of the examples here at one project, we see the castings to piles, we which are, which we call as a castings to piles or continuous flight auger. And on the left hand side, we see the bore well being made the right hand side the finished piles that are already in place.

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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 wifes 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.

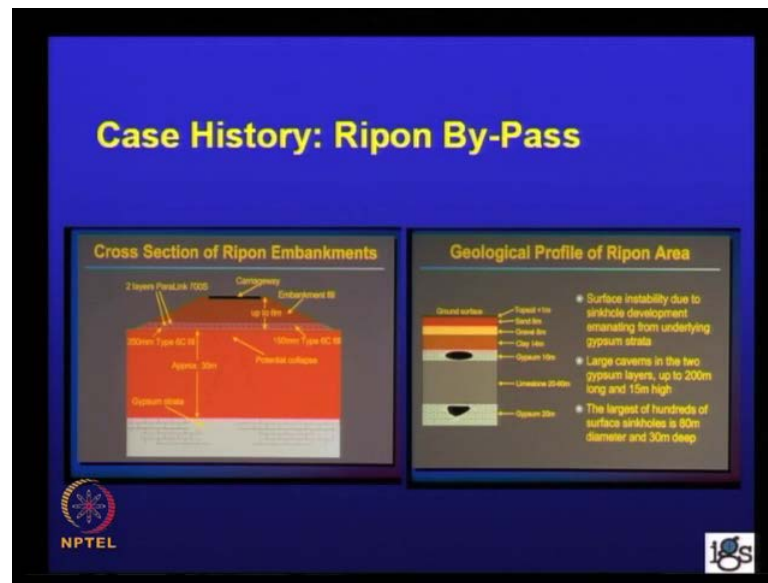
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And here, we see once again, the small diameter piles installed and then very strong geogrid.



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

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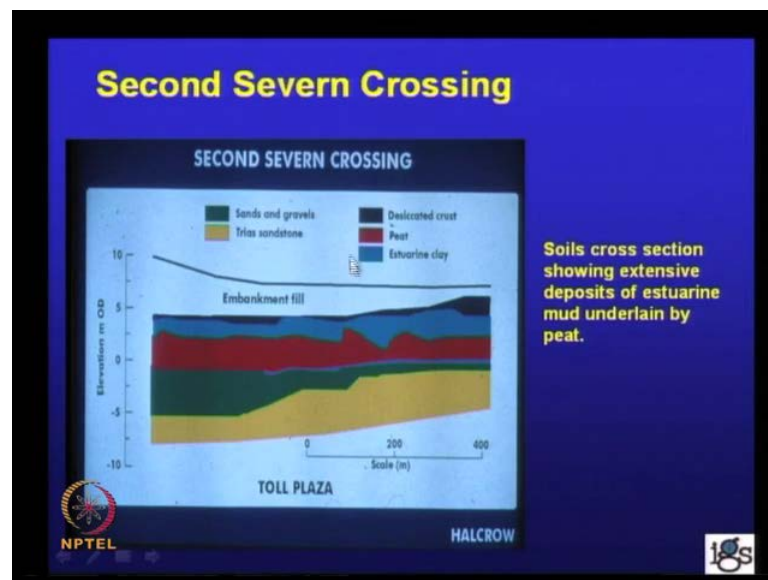
Basically, we have number of these this polymeric strips, each may be about 50 kilo Newton to 100 kilo Newton capacity, they all are linked together.

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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

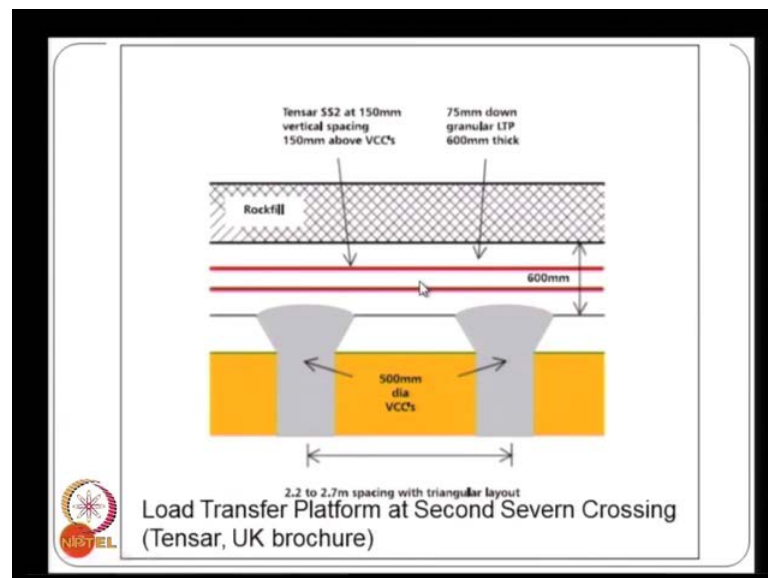
embankment construction and if it passes through that type of a soil, we should expect very large settlements and then the problems, because of lateral flows.

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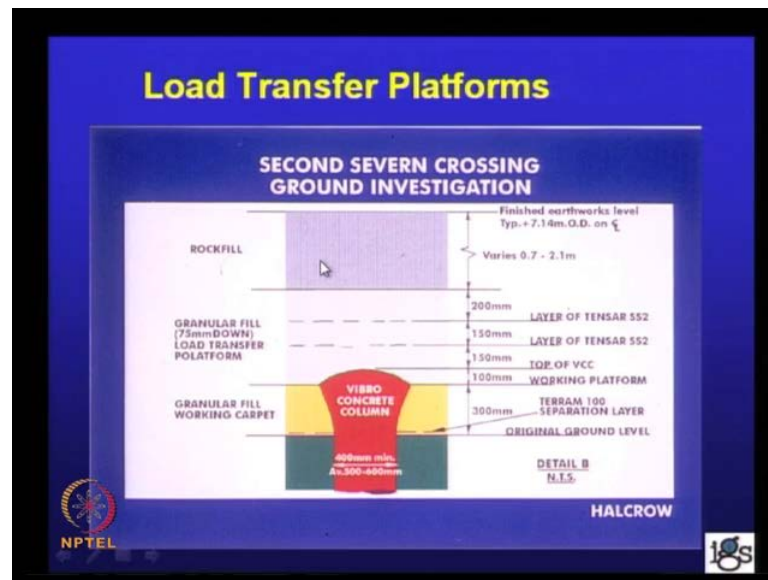
And the proposed solution was the placing the vibro concrete columns.

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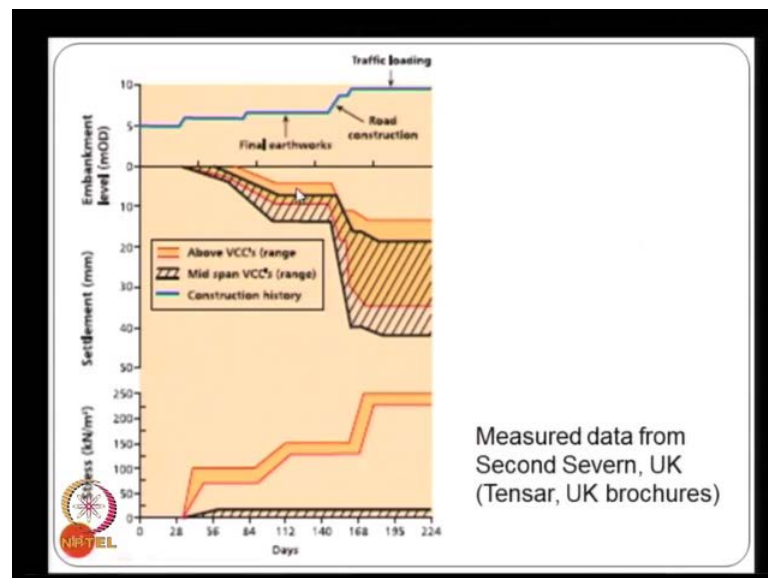
At about 2 to 2.7 meter spacing and the then that is reinforced with 2 layers of relatively weak geogrids S S 20 or S S 2, they have a tensile strength of about 20 kilo Newtons per meter.

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

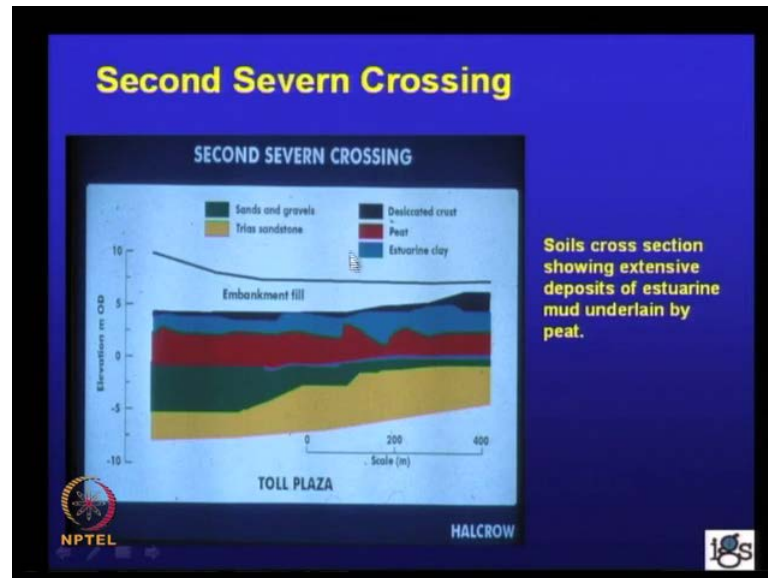
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And the performance and the monitored performance is excellent as shown here and the bottom part, we see the stress and the top part, we have the settlement is actually the with time. The construction of embankment took place for constructing about 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

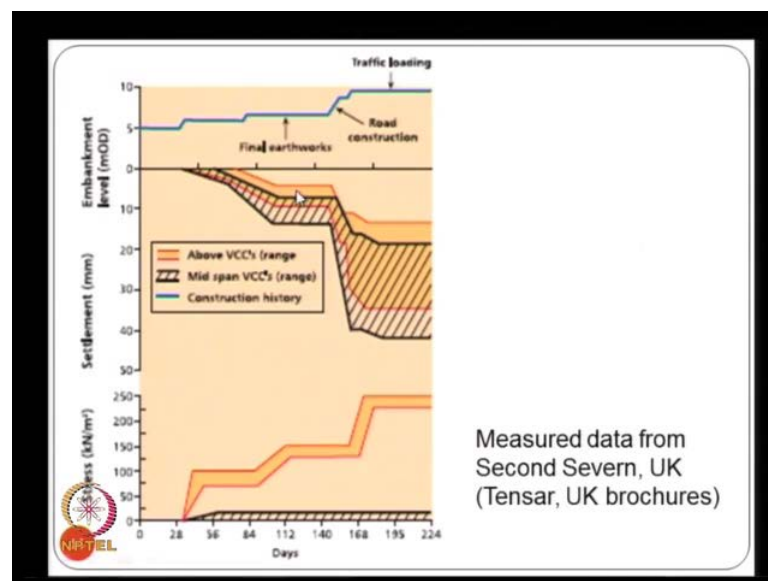
the settlements on top of these piles and the mid span between the pile supports is anywhere from about 45 millimeters to 15 millimeters. So, the settlements in spite of the fact that.

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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.

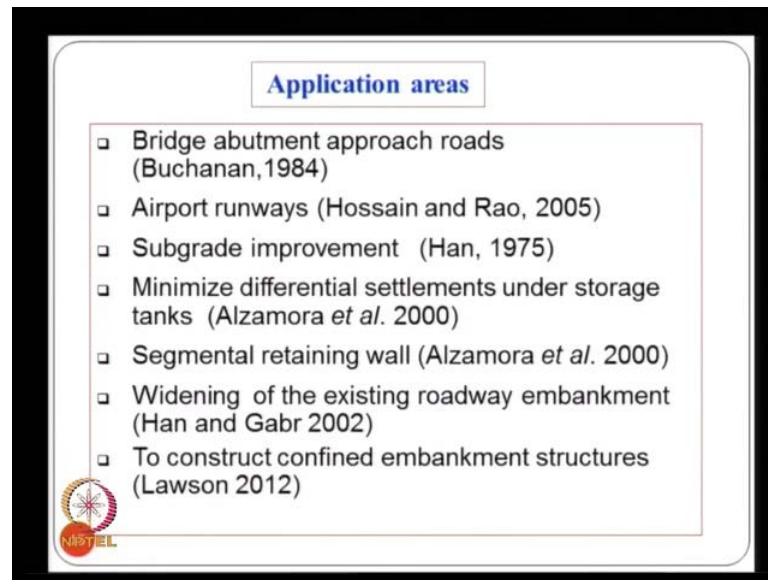
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And more importantly the bottom graph, it shows the loads that are transferred into the subsoil and the piles. And the very small amount of applied load is transferred into the

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 line, 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 widen 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

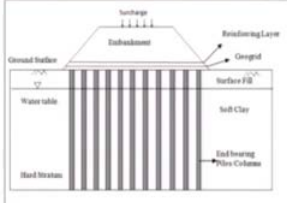


the road embankment on this type of pile supports, and of course to construction of the tall embankment structures.


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### Construction Sequence

- ❑ Installing piles with certain grid formation in the soft soil up to a certain depth.
- ❑ Geosynthetic material is laid on top of a thin layer (0.1 m) of granular material.
- ❑ After placing the geosynthetic layer, the embankment fill is constructed to the required height in stages.
- ❑ Finally the construction such as railway or road pavement is built on top of the embankment



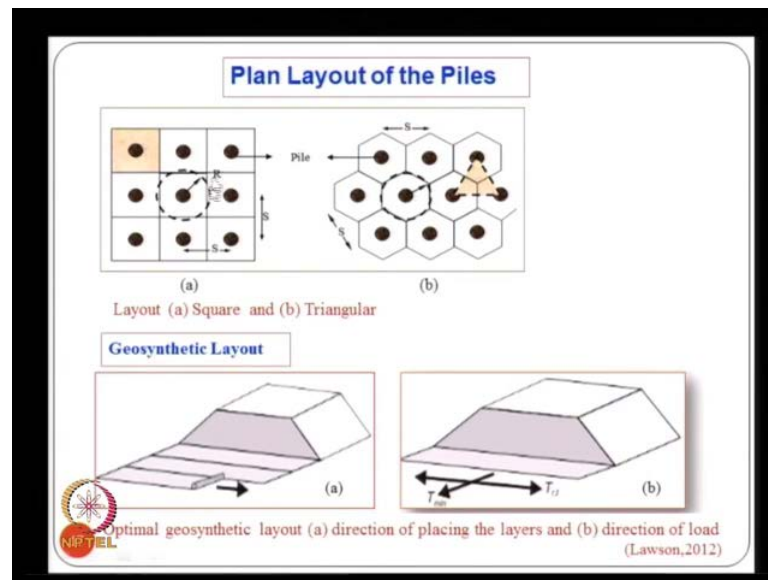
Geosynthetic Reinforced Piled Embankment System



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.

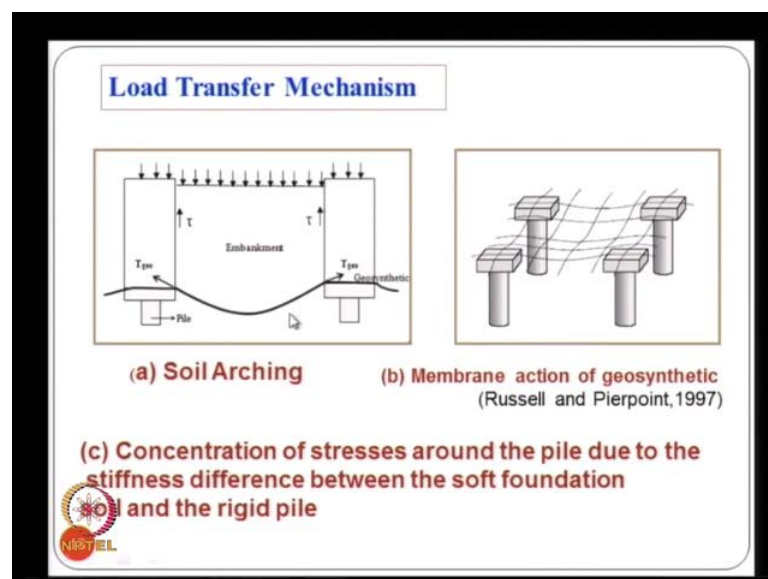


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And these are the 2 different plan layouts for the for this piles, either square pattern or a triangular pattern, that we have seen earlier with the case of the P V D's and then the geosynthetic is laid like this, the predominant load direction in 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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### Design Methods

(a) British Standard-BS8006:1995

- This is the most widely used method and is very conservative.
- Based on Marston's (1913) formula for positive projecting conduits, Jones *et al.*(1990) developed an empirical relationship for the ratio of average vertical stress acting on the pile caps to the average vertical stress acting across the base of the embankment .

$$\frac{p'_c}{\sigma'_v} = \frac{C_c a}{H} \text{ where } \sigma'_v = \gamma H$$

$p'_c$  = Arched vertical stress on top of the pile  
 $\sigma'_v$  = Average vertical stress on top of the pile  
 $C_c$  = Arching Coefficient (Marston 1913)  
 $a$  = size of pile caps



Positive Projecting Conduit  
(Marston, 1913)

And all the design methods, they rely 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.

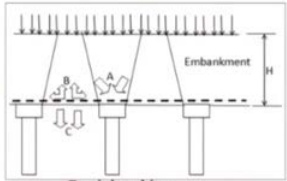
And that is equal to  $C_c$  times  $\sigma_v$  divided by the capital H, here our  $\sigma_v$  is nothing but  $\gamma H$ , that is the average vertical stress and  $p_c$  is the is the arched vertical stress, that is much higher than the average vertical stress, because of the soil arching phenomenon 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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- BS8006 adopted Jones *et al.* (1990) for the design of piled embankments.
- BS8006 gives empirical equations for arching coefficient  $C_c$  as follows
 

End bearing piles,  $C_c = 1.95 \frac{H}{a} - 0.18$

Friction piles,  $C_c = 1.5 \frac{H}{a} - 0.07$
- BS8006 considers two cases
  1. Embankment height is below the critical height of 1.4(s-a):
    - > Arching is not fully developed



Partial arching

Here A= Load acting on the piles due to arching, B= Load taken by the geosynthetic and C= Load acting on the soft subsoil

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 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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For  $0.7(s-a) \leq H \leq 1.4(s-a)$ ,

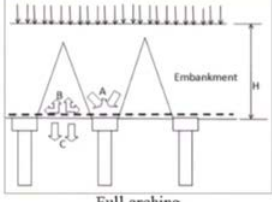

$$\text{Load on the geosynthetic, } W_r = \frac{s(f_b \gamma H + f_q W_s)}{s^2 - a^2} \left[ s^2 - a^2 \left( \frac{P_c}{\sigma_v} \right) \right]$$

$$\text{Geosynthetic Tension, } T_r = \frac{W_r (s-a)}{2a} \sqrt{1 + \frac{1}{6\varepsilon}}$$

where  $\varepsilon$  is the geosynthetic strain  
 $f_b, f_q$  are the partial factors used in the design

**2. Embankment height is above the critical height of  $1.4(s-a)$ :**

Full arching is developed

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  $s^2 - a^2$  multiplied by  $s^2 - a^2$   $p_c$  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  $2a$  square root of  $1 + \frac{1}{6\epsilon}$ , 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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- ❑ Height of embankment above arching height plays no role in the tension developed on the geosynthetic.
- ❑ Same is the case with surcharge

For  $H > 1.4(s - a)$ ,

$$W_T = \frac{1.4s f_\beta \gamma (s - a)}{s^2 - a^2} \left[ s^2 - a^2 \left( \frac{p_c}{\sigma_v} \right) \right]$$

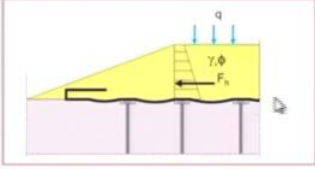
Geosynthetic Tension,  $T_r = \frac{W_T (s - a)}{2a} \sqrt{1 + \frac{1}{6\epsilon}}$



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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**Horizontal force at the slope**




Horizontal force at the embankment slope after BS8006  
(Sanbu, 2009)

□ Geosynthetic tensile load needed to resist the horizontal force of the embankment is  $T_{rs}$

$$T_{rs} = 0.5K_a(f_{\beta}\gamma H + 2f_q q)H$$

where  
 $K_a$  = Active lateral earth pressure coefficient  
 $f_{\beta}, f_q$  = partial factors used in the design



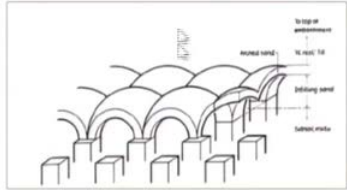
And what are the other factors that, we need to consider especially, when we are dealing with high embankments, we should see the possibility of lateral slip, that is the because we are providing relatively rigid base here, the entire soil may want to flow out by taking this as a weak plane. And this may give rise to additional forces within our reinforcement and to prevent this, we normally provide an anchor block here, that sticks 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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**(b) Hewlett and Randolph Method(1988)**

- This theory is based on limit state of soil in hemispherical domed region over piles.
- The stability of arch at the crown and at the pile top of the hemispherical dome formed defines the entire stability.



Hemispherical domes (Hewlett & Randolph, 1997)

The same arching theory was modified by hewlett and randolph and they considered a spherical type of arched surface, where as terzaghi 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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- Stress Reduction Ratio ( $S_{3D}$ ) defined as the ratio of the average vertical stress acting on the reinforcement to the overburden pressure due to the embankment fill was used to check the stability.

$$S_{3D} \text{ at the crown of the arch} = \frac{1}{\left(\frac{2k_p}{k_p+1}\right) \left[ \left(1-\frac{a}{s}\right)^{1-k_p} - \left(1-\frac{a}{s}\right) \left(1+\frac{a}{s}k_p\right) \right] + \left(1-\frac{a^2}{s^2}\right)}$$

$$S_{3D} \text{ at the pile top} = \left(1-\frac{a}{s}\right)^{2(k_p-1)} \left[ 1 - \frac{2s(k_p-1)}{\sqrt{2H}(2k_p-3)} \right] + \left[ \frac{(s-a)2(k_p-1)}{\sqrt{2H}(2k_p-3)} \right]$$

- Largest value is the critical  $S_{3D}$

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

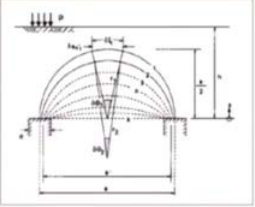


equations for the stress that at the crown of the arch and then the pressure, that is transferred into the piles and this is our including the stress intensity factor.

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(c) The new German Method (EBGEO 2004)

- In the old German approach the arching model developed by Hewlett and Randolph (1988) was used to calculate the stresses generated due to arching.
- EBGEO 2004 adopts the multi-shell arching theory based on the work of Zaeske (2001).



Multi shell arching theory adopted in New German Method (Kempfert, 2004)

The diagram illustrates the multi-shell arching theory. It shows a cross-section of a pile foundation with a load  $P$  applied at the top. The load is distributed into multiple arches, each with a different radius of curvature. The arches are shown as curved lines connecting the top of the pile to the ground surface. The diagram also shows the stress distribution on the pile and the ground surface. The stress on the pile is shown as a series of vertical lines, and the stress on the ground surface is shown as a series of horizontal lines. The diagram is labeled with various parameters such as  $r_1, r_2, r_3, r_4$  for the radii of the arches, and  $\sigma_1, \sigma_2, \sigma_3, \sigma_4$  for the stresses on the pile. The diagram is also labeled with  $\sigma_1, \sigma_2, \sigma_3, \sigma_4$  for the stresses on the ground surface. The diagram is also labeled with  $\sigma_1, \sigma_2, \sigma_3, \sigma_4$  for the stresses on the ground surface.

And the German design method is one of the more recent ones, which is a 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 a slightly different approach, that was proposed in 2001 and once again this is also this theory assumes some spherical shell type surface for estimating the pressures.

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□ 3-dimensional soil element is considered and the equilibrium of forces about the radial direction is used to calculate the vertical stress  $\sigma_{z0,k}$  coming onto the soil

$$\sigma_{z0,k} = \lambda_1^z \left( \gamma_c + \frac{p_k}{h} \right) \left( h (\lambda_1 + h_g^2 \lambda_2)^{-z} + h_g \left( \lambda_1 + \frac{h_g^2 \lambda_2}{4} \right)^{-z} - (\lambda_1 + h_g^2 \lambda_2)^{-z} \right)$$

where

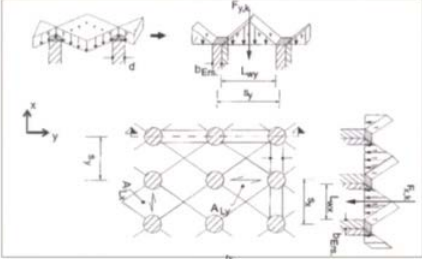
$$\lambda = \frac{a(K_{\text{stat}} - 1)}{\lambda_2 s}, \quad K = \tan^2 \left[ 45^\circ + \frac{\phi_c}{2} \right], \quad \lambda_1 = \frac{1}{8} (s - a)^2, \quad \lambda_2 = \frac{s^2 + 2a(s - a^2)}{8}$$

□ In the second step the vertical stress acting on the top of the subsoil  $\sigma_{z0,k}$  is used to calculate the vertical load  $F_k$  on the geosynthetic.




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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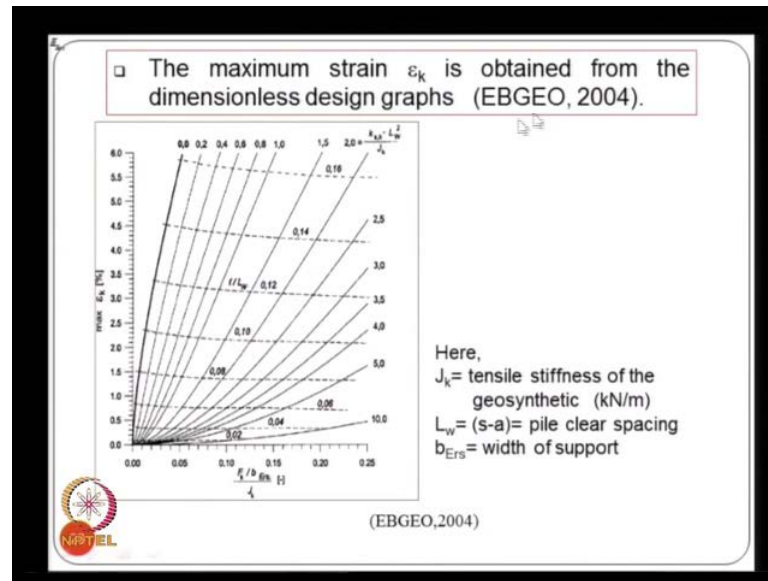
Load distribution on the geosynthetic for rectangular pile layout  
(Kempfert, 2004)

$$A_{Lx} = \frac{1}{2} (s_x s_y) - \frac{a^2}{2} \operatorname{atan} \left[ \frac{s_y}{s_x} \right] \frac{\pi}{180}, \quad F_{x,k} = A_{Lx} \sigma_{z0,k}$$

$$A_{Ly} = \frac{1}{2} (s_x s_y) - \frac{a^2}{2} \operatorname{atan} \left[ \frac{s_x}{s_y} \right] \frac{\pi}{180}, \quad F_{y,k} = A_{Ly} \sigma_{z0,k}$$


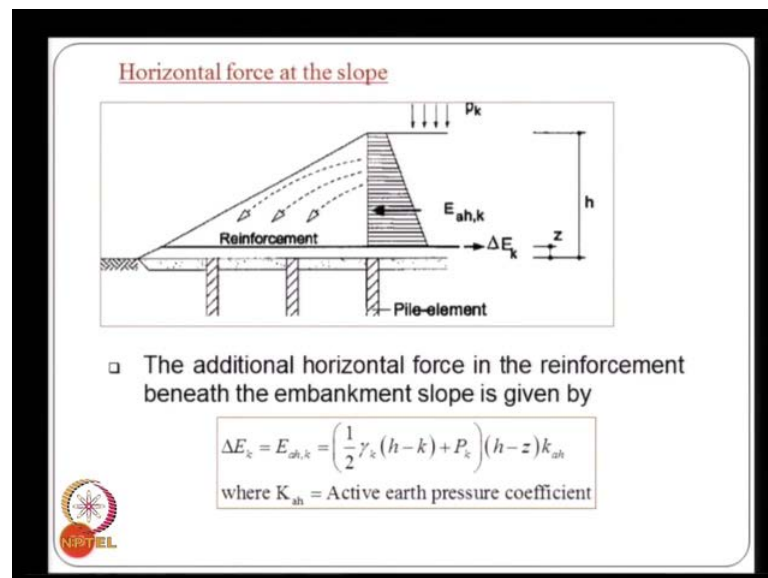
And these are some of the mechanisms, that were outlined by Kempfert, where we can calculate the force, that is transferred into the piles through these equations.

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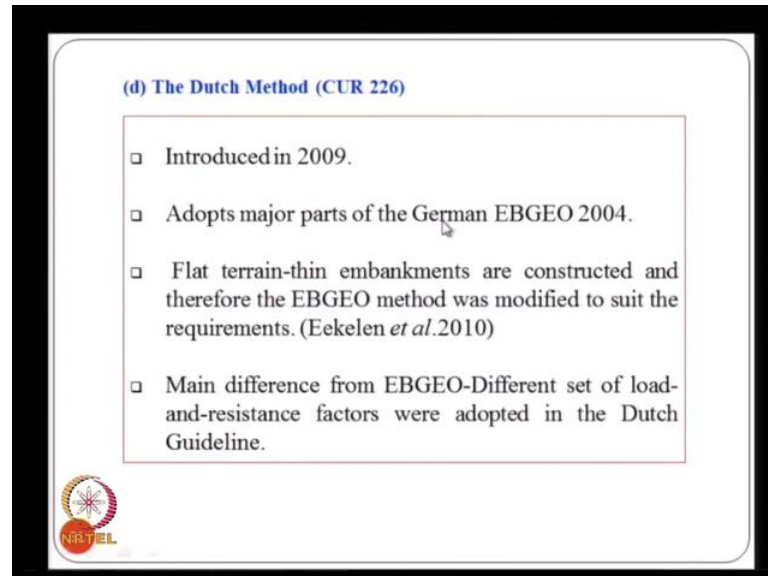
And the this is once again from the German code, the more recent one version the maximum strain developed in the geosynthetic reinforcement is expressed in terms of some factor. The  $f$  by  $b$  where our different factors  $b$  is the width of the support, that is the pile cap width and  $l$  is the 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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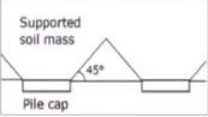


And more recently the Netherlands in 2009, they have introduced their own design methods, which is modified form of the German code. And is actually this the in Netherlands the height of the embankments are or much fatter. And then the they have adopted the German code to suit their own geometry. And 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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(e) Guido Method

- Guido *et al.* (1987) observed that the inclusion of stiff biaxial geogrid within a granular fill improved the bearing capacity of the foundation soil.
- Concluded that the angle of load spread through a granular fill reinforced with geogrid would be at an angle of 45 degrees.
- The approach is mainly for a single layer of geosynthetic at the base of the embankment fill.



Supported soil mass  
45°  
Pile cap

Stress Reduction Ratio =  $S_{3D} = \frac{(s-a)}{3\sqrt{2}H}$



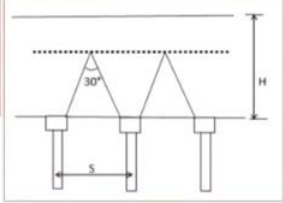
And Guido *et al.* in 1987, they also proposed some theory, wherein they assumed that instead of a domed surface, we can assume this arched portion as a triangular portion, which is at 45 degrees. They observed that the inclusion of stiff biaxial geogrid improves the bearing capacity of the foundation soil. And we can assume that the load is transferred 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  $s - a$  by 3 times square root 2 times  $H$ , that is the reduction factor, for these stresses within the soil.

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**(f) The Swedish Method**

- Carlsson (1987) considered a wedge of soil with an internal angle at the apex of the wedge equal to 30°.
- Valid in two-dimensional model.
- Carlsson adopted a critical height of 1.87(s-a).
- Miriam and George (2003) presented the expression for  $S_{3D}$  for this model as per Hewlett & Randolph (1997)

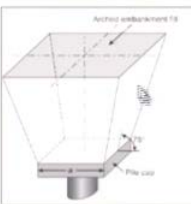
$$S_{3D} = \frac{(2s + a)(s - a)}{6(s + a)H \tan 15^\circ}$$


Two dimensional model by Carlsson, 1987

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 on 1.4 times s minus a. And their, stress concentration factor is given like this.

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- Rogbeck *et al.* (1998) modified this model into a 3D form which is an inverted truncated pyramid



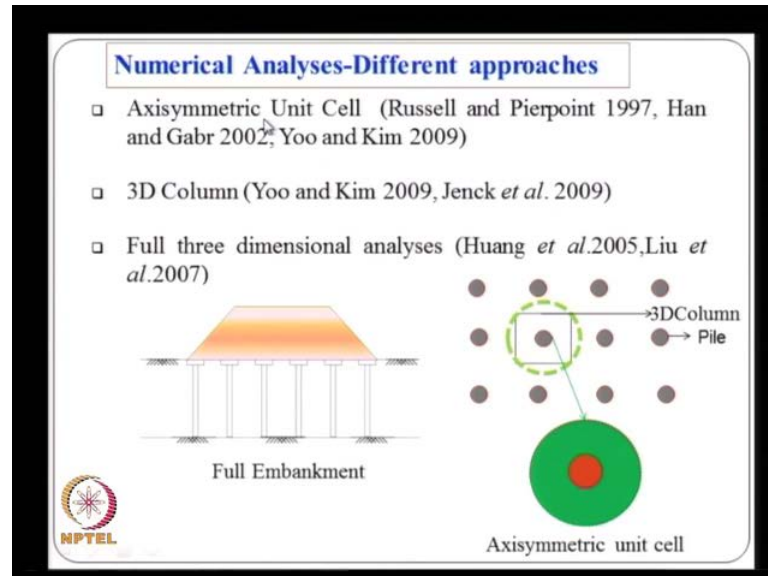
Three dimensional model by Rogbeck *et al.* ,1998  
(Lawson, 2012)

- Modified form of this 3D arching model was adopted by Nordic authorities (Svanø *et al.* 2000).

In Nordic design the arching angle was widened to include an angle of arching between 68°-75°.

And another form of looking at it is by Rogbeck, wherein in Pyramidal 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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**Major Numerical Work-3D Column**

- Russell and Pierpoint (1997) carried out a numerical study using FLAC<sup>3D</sup> to compare the different analytical methods.  
-Terzaghi (1943), Hewlett and Randolph (1988) and BS 8006
- Two cases were considered-The A13 piled embankment (heavily reinforced) and the Second Severn Crossing embankment (minimal reinforcement).
- Design methods predicted differently for different embankment geometries

Tension force calculated by different design methods

Design Methods	A13 Embankment (Reinforcement Tension, kN/m)	Second Crossing (Reinforcement Tension, kN/m)
BS8006	73	491
Terzaghi	104	297
Hewlett & Randolph	104	280

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, they 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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**Major Numerical Work-Axisymmetric unit cell**

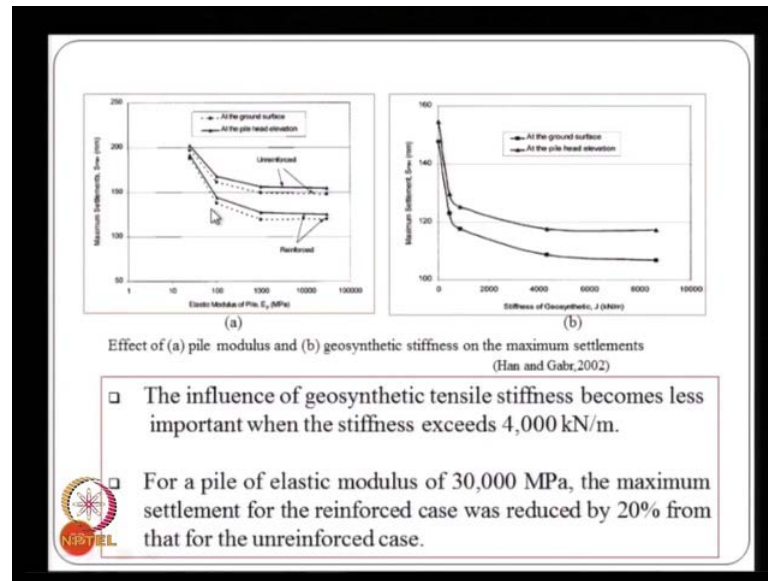
- Han and Gabr (2002) investigated the influence of the tensile stiffness of the geosynthetic, the height of the fill, and the elastic modulus of the pile material.
- One layer of geosynthetic was used and a full bond was assumed between the geosynthetic and the soil.
- Major findings are given below.

Pile Layout and the axisymmetric model considered for the analysis (Han and Gabr,2002)

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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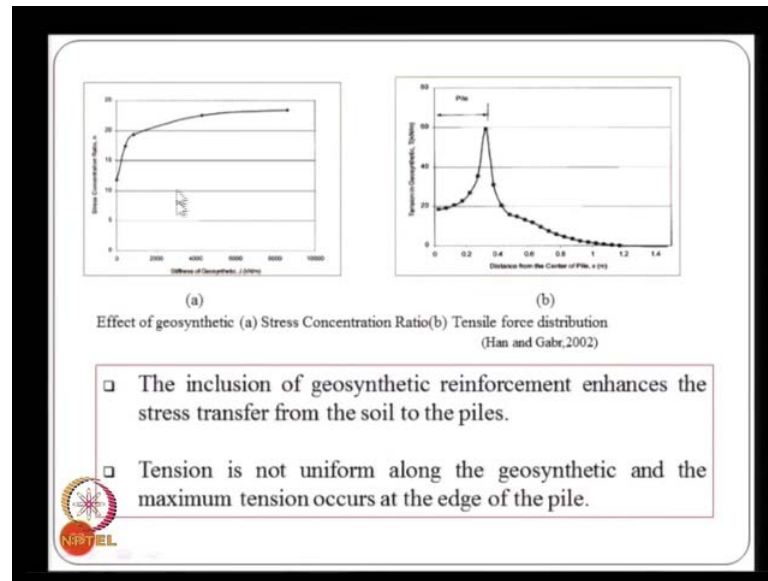


And some of their findings are like this and they have shown the relation between the elastic module of pile and then the maximum settlement, the four unreinforced case and for the reinforced case and with reinforced settlements are found to be lower. And the settlements shown with the dashed line, these 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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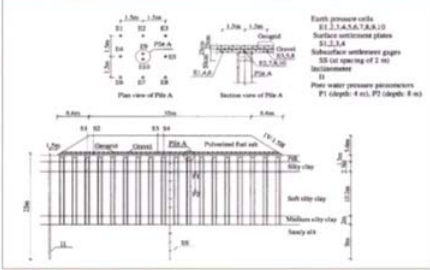
And the stress concentration factors, they have given against the stiffness of the geosynthetic, basically the stress concentration factor is the stress on the pile divided by the average vertical stress that is the  $\gamma H$ . And when we do not have any reinforcement the stress concentration is about 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 as 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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**Major Numerical Work-Full three dimensional**

- Geogrid Reinforced Pile supported highway embankment located in Shanghai China-Liu *et al.* (2007)
- Case history back analyzed by 3D fully coupled finite-element analysis.

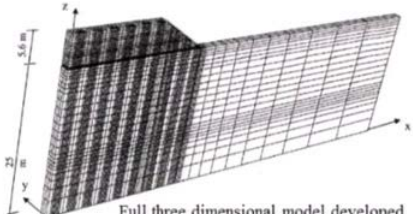


Instrumented cross section of the embankment (Liu *et al.*, 2007)

The diagram shows a cross-section of an embankment with various layers and measurement points. The layers include Earth pressure cells, Surface settlement plates, Subsurface settlement gauges, Settlement, and Pore water pressure parameters. The embankment is supported by a geogrid reinforced pile structure. The diagram also shows the plan view and section view of the pile.

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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Full three dimensional model developed (Liu *et al.*, 2007)

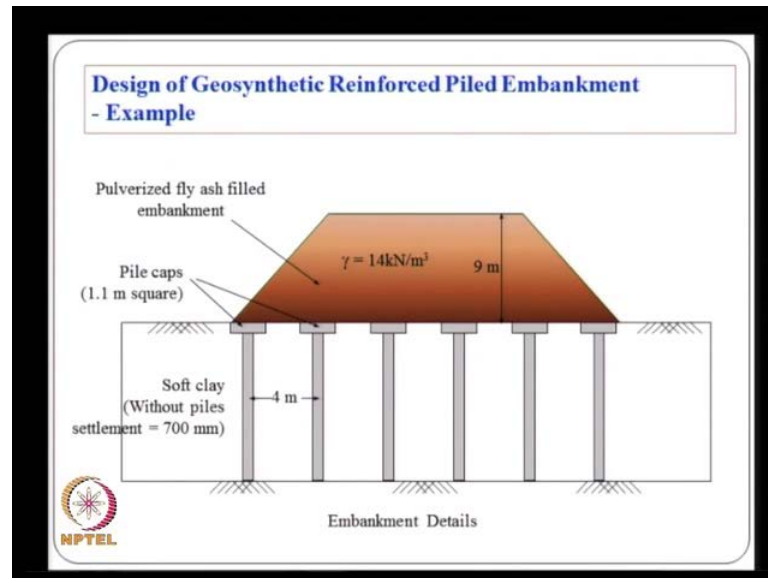
- Significant load transfer from the soil to the piles due to soil arching-contact pressure acting on the pile was 14 times higher than that acting on the soil located between the piles.
- Lateral displacements considerably reduced- stability of the embankment increased significantly.

The diagram shows a 3D model of the embankment structure, illustrating the soil arching effect and the load transfer from the soil to the piles. The model is a full three-dimensional representation of the embankment and pile system.

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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**Reinforcement details**

- Low creep reinforcement
- Tensile safety factor = 3.0
- Peak extension at failure = 12%

Geotextiles	Longitudinal Strength (kN/m)	Transverse Strength (kN/m)
A	1000	50



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.

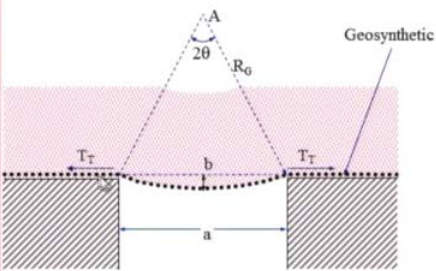
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**Circular arc Deformation analysis**


$a = 4 - 1.1 = 2.9 \text{ m}$

Assuming  
 $b = 0.2 \times 0.7 = 0.14 \text{ m}$

From the geometry  
 $\frac{b}{a} = \frac{1}{2} \tan \frac{\theta}{2}$   
 $\therefore \theta = 11.03^\circ$   
 $a = 2R_G \sin \theta$   
 $R_G = 7.58 \text{ m}$



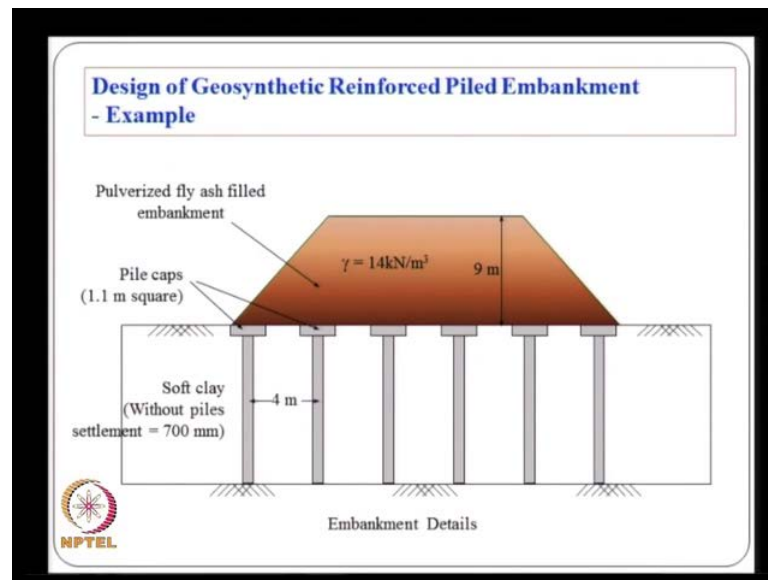
Weight of the fill,  $W_T = \frac{1}{2} \gamma (R_G - b)$   
 $W_T = 52.08 \text{ kN/m}$



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

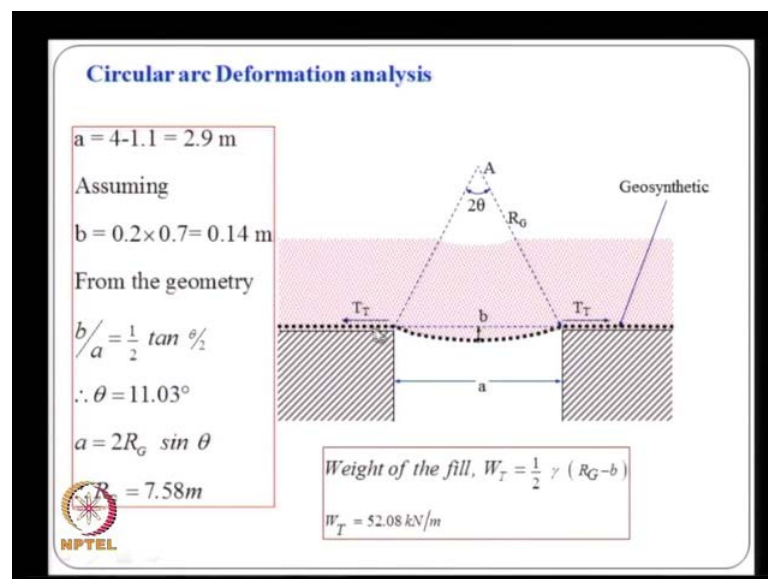


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The settlements without any treatment or 700 m m.

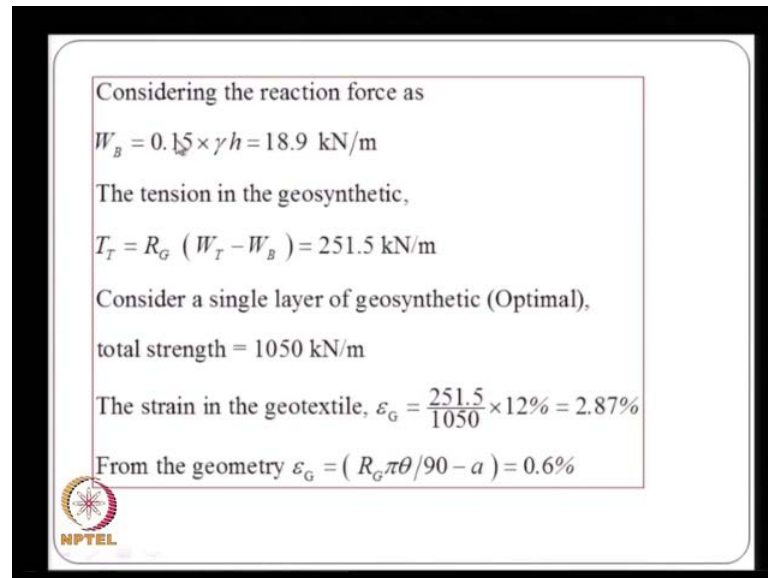
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And lets say that, because of the placement of the geosynthetic pile system our settlements are only 20 percent of the settlements of the untreated ground is about the maximum settlement is about 140 millimeters. And if you assume that, between the 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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Considering the reaction force as

$$W_B = 0.15 \times \gamma h = 18.9 \text{ kN/m}$$


The tension in the geosynthetic,

$$T_T = R_G (W_T - W_B) = 251.5 \text{ kN/m}$$

Consider a single layer of geosynthetic (Optimal),  
total strength = 1050 kN/m

The strain in the geotextile,  $\epsilon_G = \frac{251.5}{1050} \times 12\% = 2.87\%$

From the geometry  $\epsilon_G = (R_G \pi \theta / 90 - a) = 0.6\%$



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.

(Refer Slide Time: 44:56)

**Circular arc Deformation analysis**

$a = 4 - 1.1 = 2.9 \text{ m}$   
 Assuming  
 $b = 0.2 \times 0.7 = 0.14 \text{ m}$   
 From the geometry  
 $\frac{b}{a} = \frac{1}{2} \tan \frac{\theta}{2}$   
 $\therefore \theta = 11.03^\circ$   
 $a = 2R_G \sin \theta$   
 $R_G = 7.58 \text{ m}$

Weight of the fill,  $W_T = \frac{1}{2} \gamma (R_G - b)$   
 $W_T = 52.08 \text{ kN/m}$

NPTEL

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

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Considering the reaction force as

$W_B = 0.15 \times \gamma h = 18.9 \text{ kN/m}$

The tension in the geosynthetic,

$T_T = R_G (W_T - W_B) = 251.5 \text{ kN/m}$

Consider a single layer of geosynthetic (Optimal),  
 total strength = 1050 kN/m

The strain in the geotextile,  $\epsilon_G = \frac{251.5}{1050} \times 12\% = 2.87\%$

From the geometry  $\epsilon_G = (R_G \pi \theta / 90 - a) = 0.6\%$

NPTEL

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

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As  $\varepsilon_G <$  the predicted

Try with  $b = 0.19$  m


- $\theta = 14.93^\circ$
- $R_G = 5.63$  m
- $W_T = 38.08$  kN/m
- $T_T = 108$  kN/m

For this the strain  $\varepsilon_G$  from the load deformation data = 1.23%

From the geometry,  $\varepsilon_G = 1.2\%$

As these two are compatible the tension in the geosynthetic

- $T_T = 108$  kN/m.
- $\varepsilon_G = 1.2\%$



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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**Catenary Deformation analysis**

From the Equation of the catenary, the tension in the geosynthetic is given by

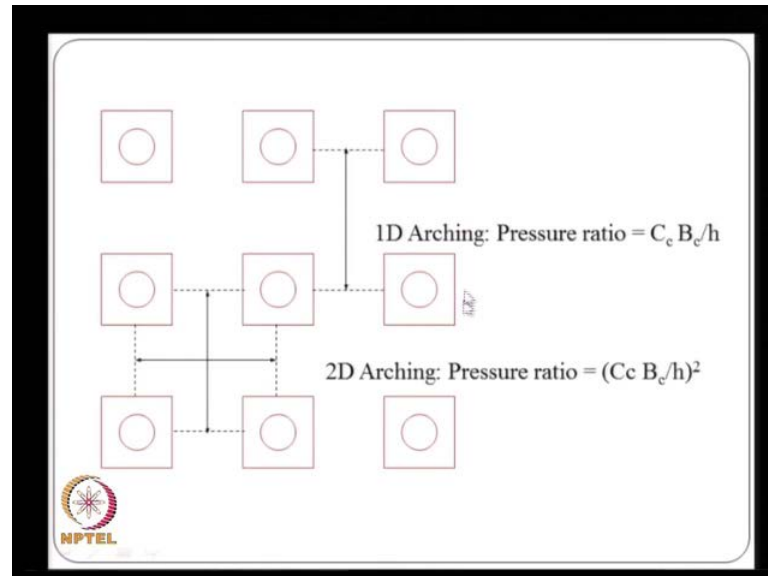
$$T_T = \frac{1}{2} (WT - WB) a \sqrt{1 + \frac{a^2}{16b^2}}$$
$$1 + \varepsilon_G = \frac{1}{2} \sqrt{1 + \frac{16b^2}{a^2}} + \left(\frac{a}{8b}\right) \log_e \left[ \left(\frac{4b}{a}\right) + \sqrt{1 + \frac{16b^2}{a^2}} \right]$$

Loading coefficient  $C_c = 1.69h/B_c - 0.12$



And we can also assume different shapes the catenary type of deformation and we can calculate under this is the arching 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  $C_c B_c$  by  $h$  square.

(Refer Slide Time: 46:42)

Loading Coefficient,

$$C_c = 1.69h/B_c - 0.12 = 13.71$$

Pressure ratio – (1D) =  $C_c B_c/h = 1.676$

Pressure ratio – (2D) =  $(1.676)^2 = 2.809$


In any 4 square piles,

- Pile area =  $1.21 \text{ m}^2$
- Total area =  $16 \text{ m}^2$
- Soil area =  $14.79 \text{ m}^2$

❖ Total load =  $16 \times 14 \times 9 = 2016 \text{ kN}$

❖ Load on the pile =  $1.21 \times 14 \times 9 \times 2.809 = 428 \text{ kN}$

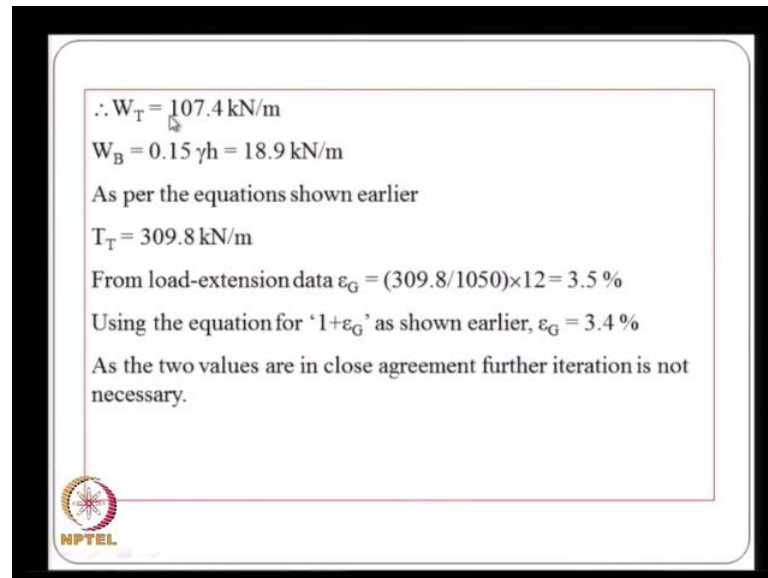
❖ Load on soil =  $2016 - 428 = 1588 \text{ kN} = 107.4 \text{ kN/m}^2$




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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$\therefore W_T = 107.4 \text{ kN/m}$   
 $W_B = 0.15 \gamma h = 18.9 \text{ kN/m}$   
As per the equations shown earlier  
 $T_T = 309.8 \text{ kN/m}$   
From load-extension data  $\epsilon_G = (309.8/1050) \times 12 = 3.5 \%$   
Using the equation for '1 +  $\epsilon_G$ ' as shown earlier,  $\epsilon_G = 3.4 \%$   
As the two values are in close agreement further iteration is not necessary.



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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**BS 8006-1995 Method**

- According to BS8006, the minimum height of embankment required is  $0.7(s-a)$  and for full arching to develop the height of the embankment should be greater than  $1.4(s-a)$
- In the present case,  $0.7 \times (4 - 1.1) = 2.03 \text{ m} < 9 \text{ m}$  and  $1.4(4-1.1)=4.06 \text{ m} < 9 \text{ m}$   
- Full arching develops in this case
- The Arching coefficient (considering end bearing pile).  

$$C_c = \frac{1.95H}{a} - 0.18$$

$$= 15.77$$
- The vertical stress on the pile cap  

$$p_v = \sigma_v \left[ \frac{C_c a}{H} \right]^2 = 14 \times 9 \left[ \frac{15.77 \times 1.1}{9} \right]^2 = 468.1 \text{ kN/m}^2$$



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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For  $H > 1.4(s-a)$ , The distributed load carried by the geosynthetic reinforcement

$$W_r = \frac{1.4s\gamma(s-a)}{s^2 - a^2} \left[ s^2 - a^2 \left( \frac{p_v}{\sigma_v} \right) \right]$$

$$= 176.85 \text{ kN/m}$$


(Serviceability condition, partial factors in the equations are given a value of 1)

Tension in the reinforcement (BS8006-Design strain is 5%)

$$T_r = \frac{W_r (s-a)}{2a} \sqrt{1 + \frac{1}{6\varepsilon}} = 486.2 \text{ kN/m}$$

Tension due to lateral thrust,  $T_L = 0.5 K a \gamma H = 170.1 \text{ kN/m}$

Total tension = 656.3 kN/m




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



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**Results of Design**

- By Circular arc method  
 $T_T = 108 \text{ kN/m}$ ;  $\epsilon_G = 1.2 \%$ ;  $W_T = 38.08 \text{ kN/m}$
- By Catenary deformation method  
 $T_T = 310 \text{ kN/m}$ ;  $\epsilon_G = 3.4 \%$ ;  $W_T = 107.4 \text{ kN/m}$
- By BS 8006 1995 method  
 $T_T = 656.3 \text{ kN/m}$ ;  $\epsilon_G = 5 \%$ ;  $W_T = 176.85 \text{ kN/m}$




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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**SUMMARY**

Geosynthetic Reinforced Piled Platforms are suitable for supporting high embankments without settlement or bearing capacity problems.

They have been employed successfully at several projects.



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.