Soil Mechanics
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Lecture – 54
Earth Pressure Theories – V

Welcome to lecture number five on earth pressure theories. In the previous lecture we have learnt about coulomb's theory of earth pressure and then we discussed about merits of coulomb's theory over Rankine's theory and then we also discussed about culmann's graphical method for computation of earth pressure.

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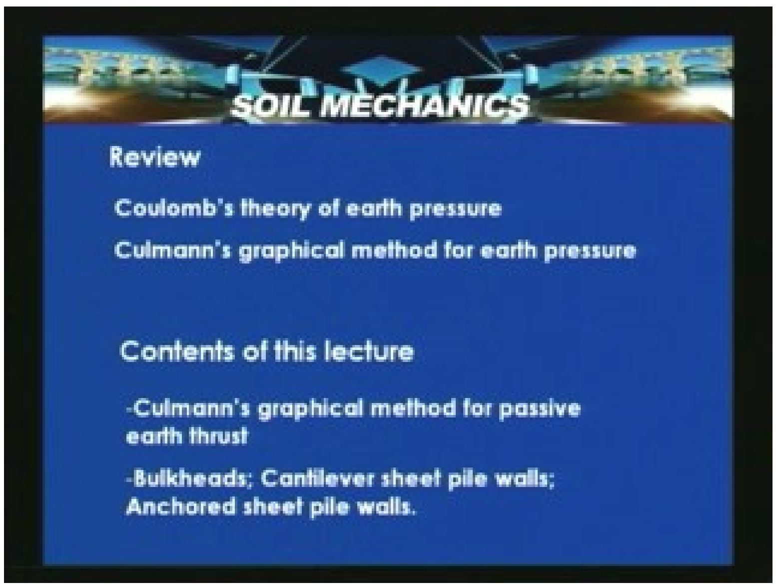

In this lecture we look into how we can estimate passive thrust based on culmann's method and applications of earth pressures. One of the important and prominent applications is in the design of bulk heads. The bulk heads basically two types cantilever sheet pile walls and anchored sheet pile walls. So in this lecture we try to cover or look into these aspects. So as discussed in the previous lecture we discussed that culmann's method can be used for determining active or passive earth pressure thrust and basically this is a graphical method.

So what we need to do is that first we need to represent the wall to a scale and ground surface and any surface like in this slide which is shown here, where you have got range of surfaces like AC_1 and C_1 to C_2 with another slope C_2 to C_3 and C_3 to C_4 and C_4 to C_5 and in case if there is some railway track or a boundary wall which is running very close to the retaining wall then effect of this external load on the earth pressure can also be determined very easily by using culmann's method.

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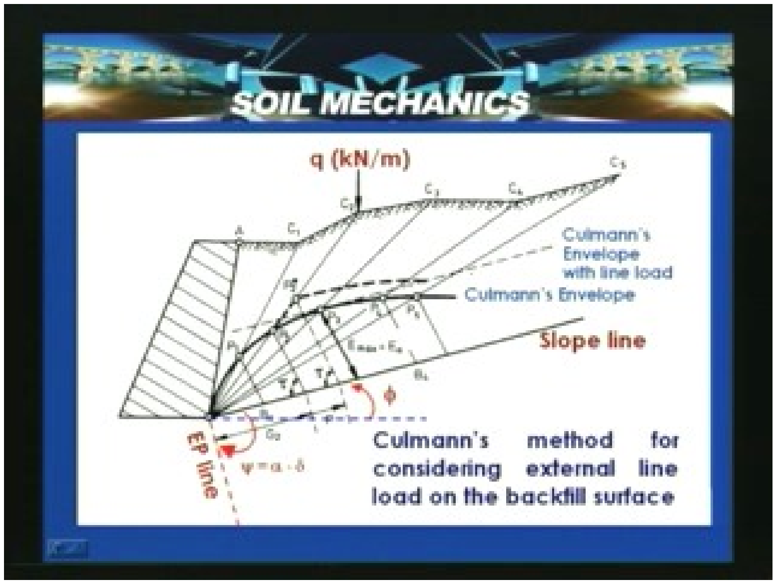

So this slide shows a method for considering external line load on the backfill surface. So what we need to do is that we need to represent this particular wall to a scale and we need to draw a slope line with an angle phi then you need to draw earth pressure line with an angle psi is equal to alpha minus delta from the slope line. Once we draw then along the ground surface to A C_1, C_2, C_3, C_4 and C_5 we need to draw the lines passing from the heal of the wall to the ground surface. So these are nothing but wedge lines, possibly one of these lines can become a potential failure surface. So here when you look we have drawn lines like BC_1, BC_2, BC_3, BC_4, BC_5 passing through the heal of the wall, B is the point here. Then once we have done we have found out the weight of this wedge ABC_1 then they represent the weight of this wedge along the slope line to a suitable scale here.

Similarly determine the weight of the wedge A B C_2 and determine its scale and then put it on the slope line. Once we do, we will be able to represent this wedge weights along the slope line. In case if there is a external load coming add this load at this point here to these particular point of wedge so then what will happen is that, this is that envelope which we are going to get after considering the external load and this is the envelope culmann's envelope without considering the external load.

So once after plotting we need to draw a line parallel to the earth pressure line and it should cut at that point where BC_1 is there and that is the point P_1 on the envelope. Similarly P_2 on the envelop P_3, P_4, P_5 can be identified and drawing joining this P_1, P_2, P_3, P_4, P_5 will give the culmann's envelope. In this figure what we saw is a culmann's envelope which is drawn for without any load and this is P_1, P_2, P_2 dash and then these corresponding points they represent the culmann's envelope with a line load. So this is how we need to consider the external loads coming on to the picture.

Now let us look into how we can consider this culmann's method for the passive case. So we look into the animation here, this is the case for passive case and with a positive wall friction. So this is indicated and wall face is vertical having a wall face interface friction is about delta then the weight line or the slope line is drawn at phi below horizontal then earth pressure line is drawn at an angle from the slope line which is psi is equal to alpha plus delta. Then we now try to draw the trial wedges. So we have drawn the first wedge line passing through the heal of the wall. So the procedure is again same here in this case the wall is actually moving towards the fill. Now what we need to do is that we have to compute the weight of that wedge and we have to represent that weight along this slope line. So that's what we did and we have drawn a line parallel to the earth pressure line.

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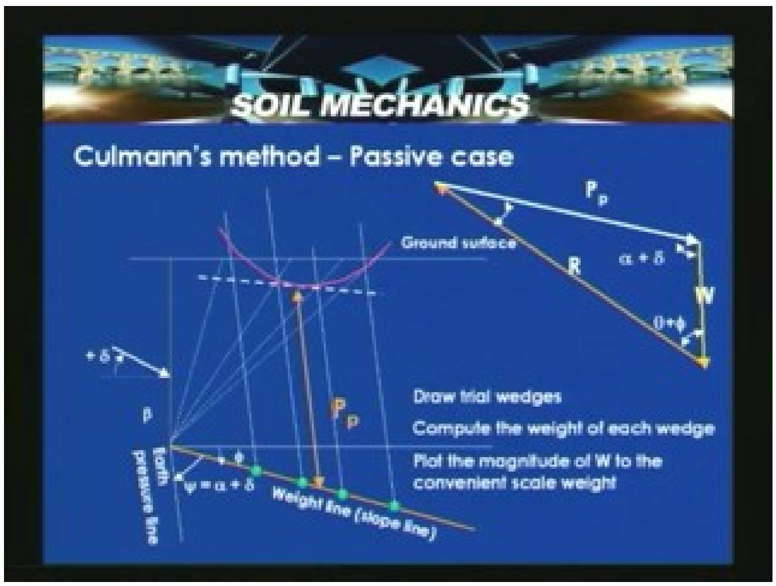

So this is the point where it cuts so this is one of the points on the possible culmann's envelope for passive case. Now similarly along the second line when it goes and it meets at this particular point. Similarly third wedge line, fourth wedge line, fifth wedge line when we look into this all these points can be joined and then this envelope this pink one which represents is the culmann's envelope and line drawing parallel to this slope line will actually give the one which is passing through this, that is this particular one will give the minimum force that is which exerted on the wall that is P_p can be termed here as passive earth pressure thrust.

Similarly culmann's method for the passive case can also be determined. In this case a wall cross section is shown and when the ground surface above this point when the wall tries to move this direction. So this portion becomes the active side and this portion becomes the passive side and when the wall is moving towards this portion will be subjected to compression. So in this case we are interested in determining the passive thrust exerted by the fill opposite to the wall, when the wall is moving towards it. So in this case a wall cross section is shown and which is represented here and a positive wall friction is considered here and P_p is the one which we need to determine.

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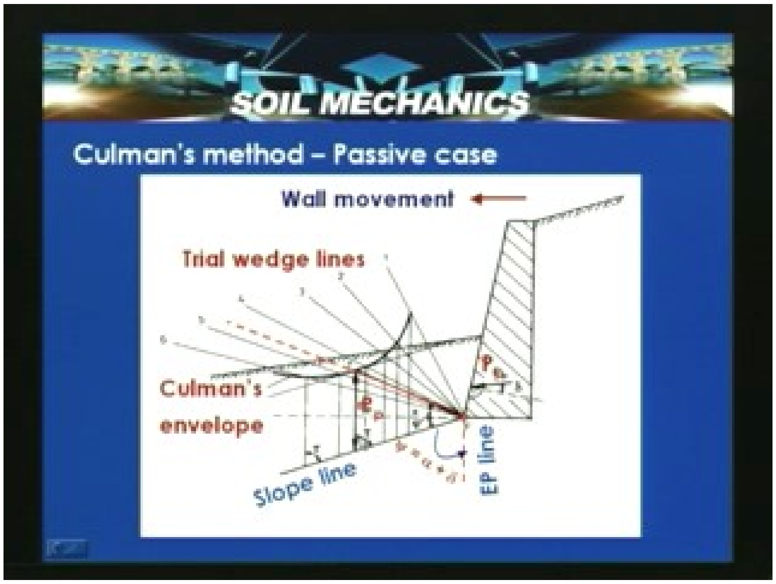

So this is the heal of the wall so what we have to done from this heal point and with a horizontal at an angle phi we have drawn a slope line and from this slope line we have drawn an earth pressure line which is at psi is equal to alpha plus delta. Then what we did is that we have drawn number of lines passing through the heal 1, 2, 3, 4, 5, 6 these are the possible wedge surfaces because we do not know which one will give the minimum earth pressure thrust on the wall. So to find out that what we do is that we have taken this and then we have found out the weight of this wedge and the weight of this wedge is represented with a suitable scale along the slope line then we have drawn a line parallel to the earth pressure line and then we have noted a point where it cuts.

Similarly repeat this exercise for the other points also then what we get is that we generate culmann's envelope. Once we draw a line parallel to this surface we can get this point where the earth pressure is minimum that is P_p that is actually is the passive earth pressure thrust offered by the wall when the wall is actually moving towards it. This is only for the portion of soil below this zone. We can say that this is the possible wedge which is actually inducing this force P_p. Now P_p is the force passive earth pressure thrust which is actually determined. Once we know this P_p then we will convert in to suitable scale, we will be able to get in kilo newton per meter.

So by adopting this procedure we can determine this by using culmann's method either active earth pressure thrust or passive earth pressure thrust. Now let us look into some features of coulomb's active and passive pressure cohesions. In this case for different wall friction angles and friction angles of the soil, we are actually given here coulomb's active pressure cohesions and coulomb's passive cohesions. When a special case like we discussed with delta is equal to zero and friction angle is equal to 33 here the coulomb's active coefficient will be equivalent to Rankine active coefficient. So the coulomb's solution can be converged into Rankine's solution if those Rankine's wall conditions are satisfied.

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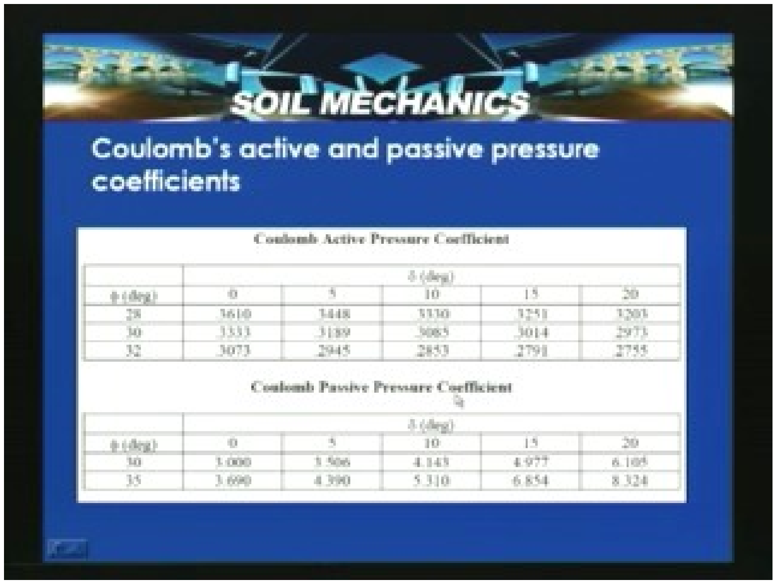

Let us look into this deliberations based on this exercise like for the coulomb case shown in the previous slide with no soil wall friction that is delta is equal to zero and a horizontal backfill surface both coulomb and Rankine methods yield equal results. That's what we said that k_a is equal to k_{ac} is equal to k_a Rankine is equal to 1 by 3, k_{pc} is equal to k_p is equal to 3. As the soil become stronger that means denser and the friction angel increases so the active pressure coefficient decreases resulting in a decrease in the active force and the passive pressure coefficient increases resulting in an increase in the passive force.

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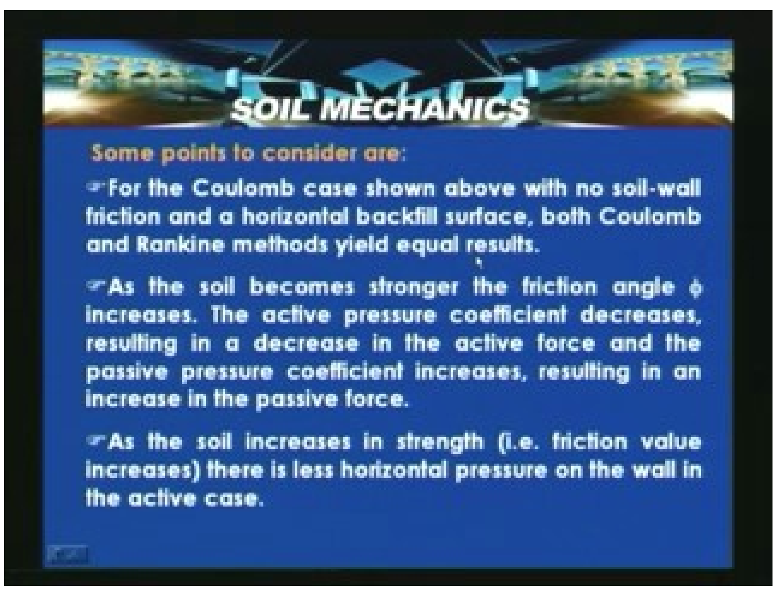

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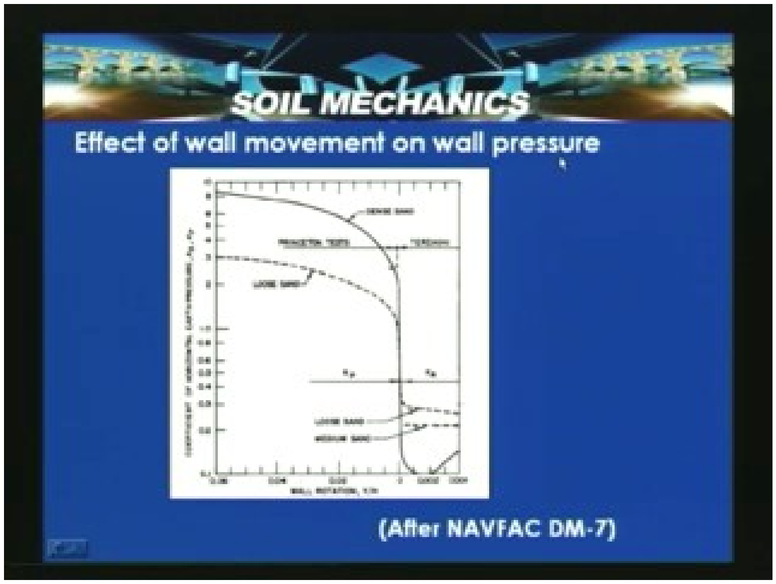

This we need to note, as the soil increases in strength that is friction value increases there is a less horizontal pressure on the wall in the active case. This indicates that you need to compact the soil so that it induces less pressures. In case of an active pressure it will be exerting a less pressure. This is after NAVFAC design manual-7 where it shows effect of wall movement on the wall pressure, this is the wall rotation and this is the coefficient of horizontal earth pressure k_a k_p. You can see this is how it varies for loose sand and this is the zone for the active case and this is the zone for passive case and this is how for dense sand.

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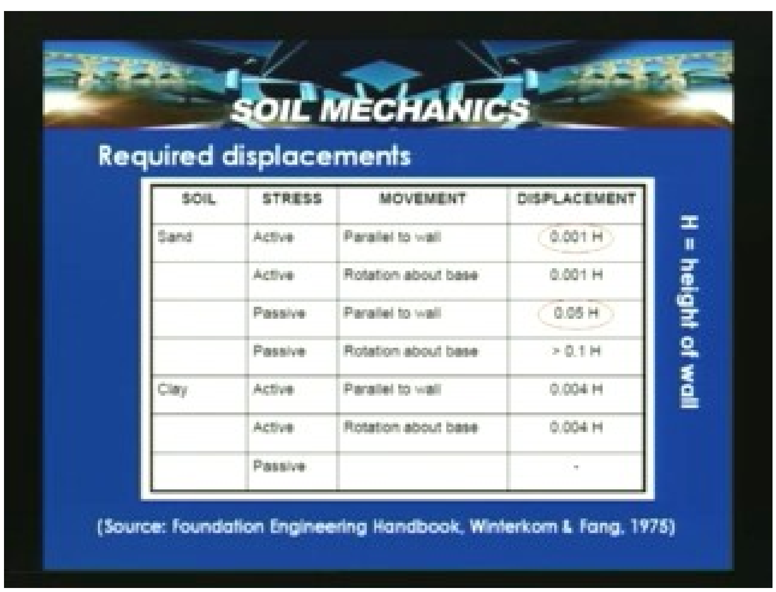

SOIL	STRESS	MOVEMENT	DISPLACEMENT
Sand	Active	Parallel to wall	0.001 H
	Active	Rotation about base	0.001 H
	Passive	Parallel to wall	0.05 H
	Passive	Rotation about base	> 0.1 H
Clay	Active	Parallel to wall	0.004 H
	Active	Rotation about base	0.004 H
	Passive		-

This we already discussed, this is shown for reference in the diagram form and also some required displacements like for in sand and clay surface and in case here active case and wall movement parallel to the wall then the displacement required is 0.001 times H where H is the height of the wall. In case the passive parallel to the wall then 0.05 times the H that displacement is required.

Similarly in case of clay, parallel to the wall 0.004 times H and rotation about the base then it also requires the similar amount of displacement. General range of wall friction angles because we have a range of fill materials which can be used, the content which is regarding the type of fill material, how it can be selected is beyond the scope of these lectures but we need to have a freely draining back fill so that the pore water pressure under control all the time.

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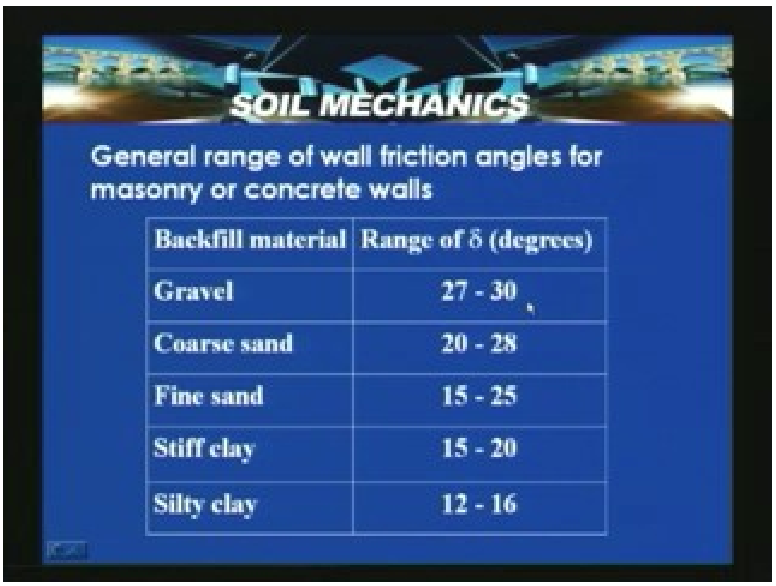

Here the typical fill materials like gravel can have a range of delta that is interface friction angle is about 27 to 30 degrees. In case of coarse sand it is around 20 to 28, fine sand 15 to 25, stiff clay 15 to 20 and silty clay it’s about 12 to 16 degrees interface friction angle. This table presents here, the general range of wall friction angles for masonry or concrete walls. Now let us look into the application of whatever we have learnt and first we said that these earth pressures can be used for determining the earth pressures exerted by the soil on the wall in active condition and passive condition and ultimately these earth pressures can be used for designing this retaining structures. We discussed that many types of retaining structures are there in the first lecture under earth pressure theories. We have introduced the different types of walls which can be possible and which can be constructed but commonly the walls which can be designed like gravity walls and gravity masonry retaining walls or reinforced concrete retaining walls, all these things which require a computation of earth pressure and based on that the moments are estimated.

Then wall stability from the sliding and overturning and from the sensitive point of view can be considered and designed. But let us look into what are these bulkheads, how these bulkheads can be analyzed for a riving at their design or whether they are stable or not. How they will deflect in case if they are anchored with anchor or without anchor. So bulkheads of the most important type of retaining structures after retaining walls and these are basically used in water front application, basically to which these are used in water front applications and bulkheads are sheet pile walls which are made up of number of thin vertical elements called sheet pile walls.

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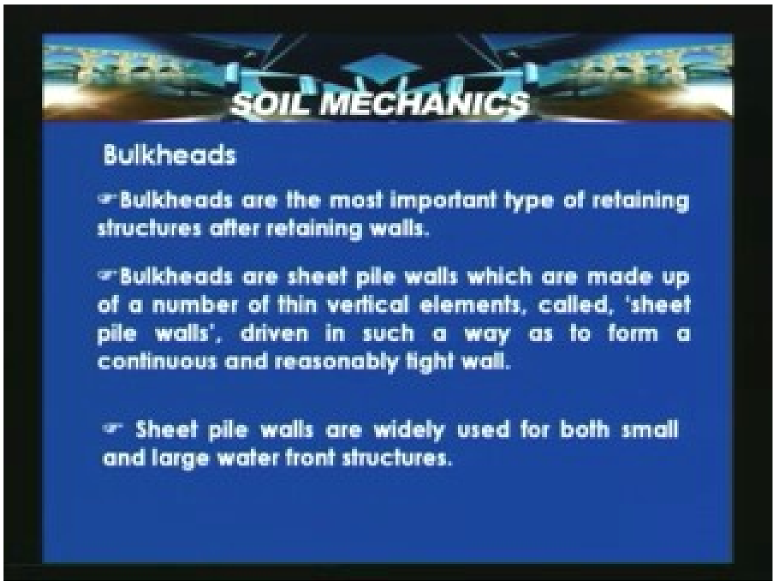

The standard sheet piles sections are available and these are driven in to the ground and connected such a way that it forms a continuous wall. The wall is very thin and it has got range of section modulus which is possible. So a typical wall or a typical section can be selected based on the section modulus which is required for a cause and also there are walls like diaphragm walls which can be constructed by digging a trenge and placing a reinforcement and introducing concrete. So bulkheads are sheet pile walls which are made up of a number of thin vertical elements called sheet pile walls driven in such a way, as to form a continuous or reasonably tight wall and sheet pile walls are widely used for both small and large water front structures.

So these are also used for in some cases like as a part of the berthing structures in the marine environment also. So in this slide a typical sheet pile wall is shown where these are marked for driving into the soil. Sometimes these are also used for protecting temporary excavations like here what you are seeing high rays building. So to protect any danger or to prevent any collapse of the soil in front of these particularly, these sheet pile walls which are driven into the ground to retain the soil behind these particular section.

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This is installed sheet pile wall, you can see a case where how the sheet pile wall are installed here with serve also as a wall for retaining this portion of the soil as well as to maintain water tightness because of the rubber interlocks as well as to have a continuous retaining wall.

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So these are also being in flexible in nature they are also called flexible retaining structures. So from here onwards we are trying to look into different types of flexible retaining structures and how these flexible retaining structures can be analyzed, what are the salient aspects in the design that we look into it.

Basically there are two types one is cantilever sheet pile walls and other one is called anchored sheet pile wall or anchored bulkhead. Cantilever sheet pile wall is nothing but which is having a section which is shown here driven into the ground and H is the retaining height and D is the embedded depth and this line is called dredge level. So the stability of the wall depends up on the embedded depth which is provided in front of the wall. So it derives a stability based on its embedded depth which is there in front of it.

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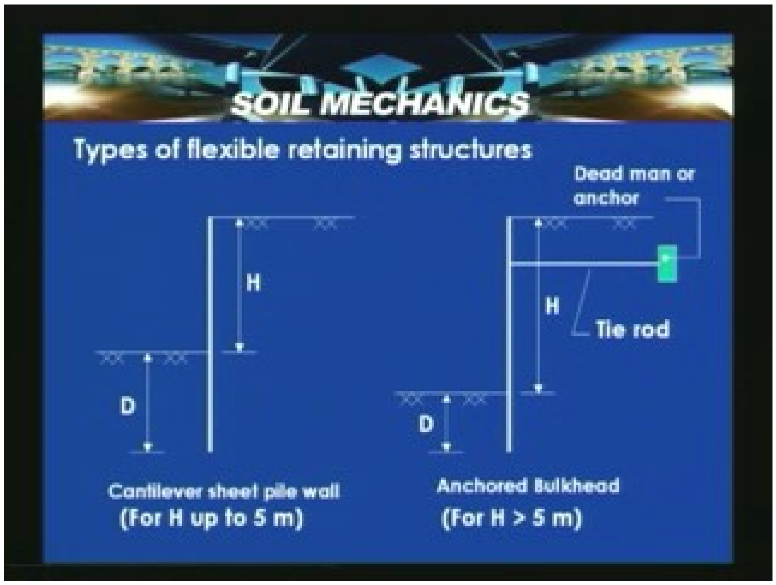

So particularly in case where water environment is there, it is required to be checked from the scour point of view. Otherwise they can endanger the stability of a structure. Sometimes the wall heights are more than 5 meters, these are actually used up to 5 to 6 meters but in case height of the retaining height is say more than 6 meters then to manage with the given section they are anchored by using an anchor plate or a anchor sheet pile wall or with battered piles. They are actually anchored along the length of the wall and these particular portions which is called tie rod. So these tie rods they even have with high tensile rods which have up to 100 mm diameter also are being used in the practice.

So here also the H is the one which is the retaining height and D is the embedded depth. The design involves finding out earth pressures and estimating D and estimating the anchor forces so that the design sections can be like what section to be provided for these place and what is the diameter of the tie rod to be provided and what will be the safe location of this tie anchor plate or a dead man anchor has to be seen. So this actually has to derive its support, it’s because of the net resistance earth pressure resistance when this is moving this side. Here it is possible that there is active case and here it is possibly that there is a passive case so that net pressure actually serves as an anchorized capacity for this case. So we introduced in this slide like two types of flexible retaining structures one is cantilever sheet pile walls and anchored sheet pile walls.

Let us look into some features of these cantilever bulkheads or cantilever sheet pile walls or anchored sheet pile walls or anchored bulkheads. In the cantilever bulkheads basically as we discuss they derive the support from ground penetration that is the depth which is embedded and an effective embedment length must be sufficient to prevent overturning.

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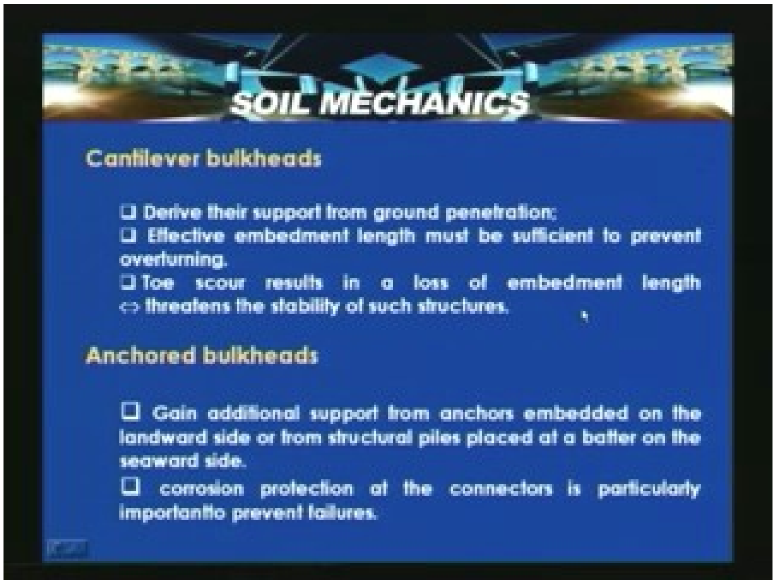

So the embedded depth should be sufficient to prevent any overturning if it is inadequate it is possible that the overturning can occur and toes scour results in a loss of embedment depth and three times to the stability of such structures. So if there is a need scour in front of the wall and that causes a loss of the embedment depth and causes three times to the stability of such structures and anchored bulkheads basically gain additional support from anchors embedded on the landward side or from the structural piles placed at a batter on the seaward side.

These cantilever sheet pile walls also can be propped externally, they are also called propped walls or propped cantilever walls or if you are using anchors within the landside or a soil side then these thyroids can be connected at and placed at suitable location. So that they have an adequate anchorized capacity. Corrosion protection at the connectors is particularly important to prevent failures.

So before looking into these design aspects, let us look into how this cantilever sheet pile walls can fail. We discuss that basically two types of failures are possible, one is that soil failure the other case material failure or the section failure whichever has been provided to retain the earth. In this case failure due to the rotation about point A, in this case the wall or the section is adequately designed but it is subjected to a rotation about a point A. That means the wall is rigid and undergoing rotation about certain point A form its tip. Now here D is the embedded depth and H is the retaining height and this is the point of rotation.

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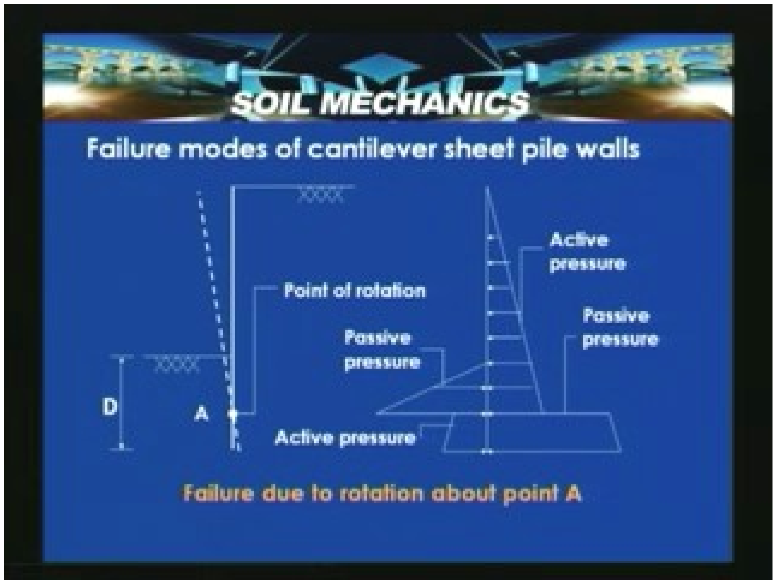

In such situation when the wall say deflects away from the surface and this zone becomes active but when you come to this point of rotation and it is below and this portion of the wall is pushing the wall inside. So here this portion becomes the passive zone and again this portion becomes active zone because in this portion the wall is going away from this soil and in this portion here you are seeing that the wall is actually coming and pressing the soil in front of the wall or just below this dredge line. In such situation and this is called the passive zone, active zone, passive zone and active zone.

So when we draw the earth pressures by using the fundamentals whatever we discuss and here in this case basically Rankine's conditions are assumed to be valid and like Rankine's coefficients are used for calculating the coefficient of active pressure or coefficients of passive pressure. So here when you draw an earth pressure diagram what we can say is that this is the active pressure which we get and this portion is the point of rotation which is shown here and this is the zone where the passive pressure is there and this is the zone where the active pressure is there and this is the zone where the passive pressure develops. In case if there is a surcharge then the additional pressure due to the surcharge will come into the picture with multiplication of k_a factor.

So this is the earth pressure diagram for this case which is shown here, in this case the wall subjected to rotation, when actually a soil failure arranges and a rotation is allowed. So this is one of the types of the failure modes of the cantilever sheet pile wall, the other failure is that as we just discuss is that formation of a plastic hingered point that means there will not be any failure or rotation about this point. The wall yields about this point and attains the moment which is beyond its plastic moment capacity of that particular section. So this is possible when suppose if there is a possible enhancement in the earth pressure and section is not adequately designed then material failure of the wall can occur and can yield this type of failure.

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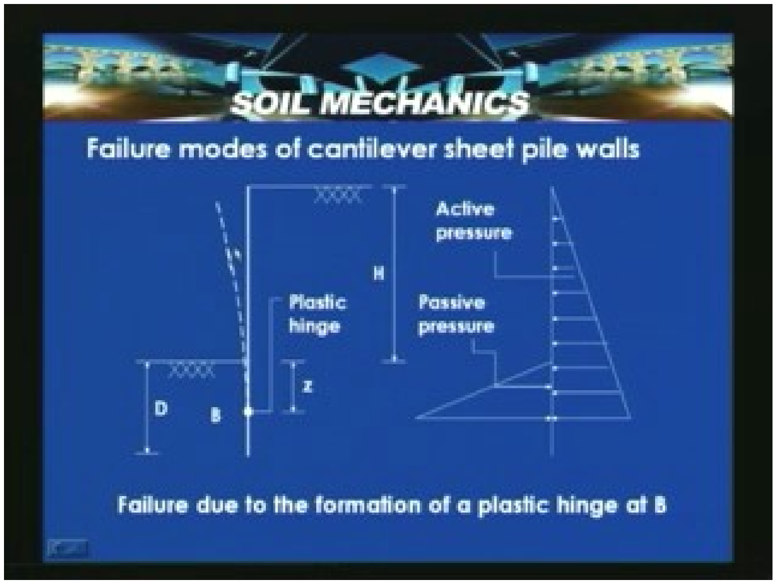

So failure due to formation of plastic hinge at B which is shown here, in such situation here the wall is either moved towards the soil or away for the soil, it remains intact. So where that no pressure will be there in both the sides so the zero pressure and here this portion again is passive pressure and this portion is again entire zone is active pressure. So this is the earth pressure which is acting at this particular point and it is actually undergoing moment about this point that is actually here the maximum bending movement will be concentrated and for these walls when we compute the bending movement, the maximum bending movement will occur below its dredge level.

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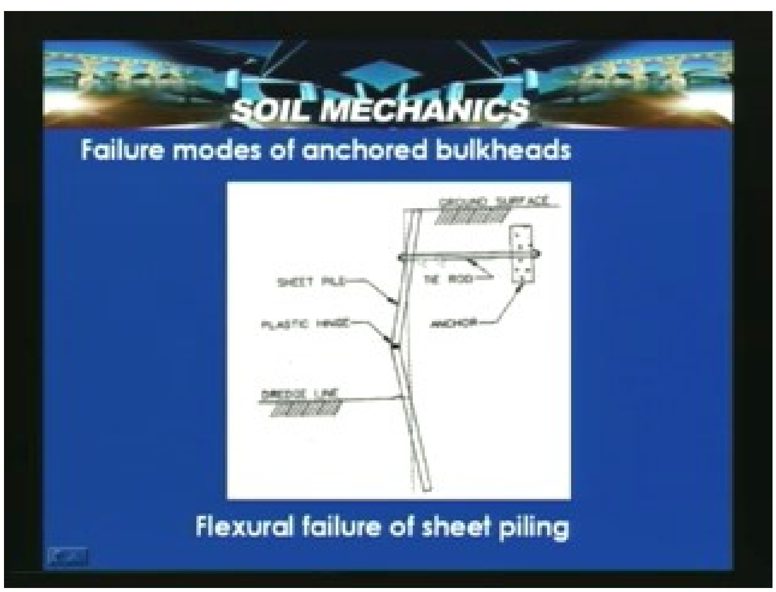

So the failure modes of the cantilever sheet pile walls and we have seen two types of failure, one is due to soil failure where the rotation can occur, the other one is the material failure where the formation of a hinge can occur. In the previous slides we have seen failure modes of the cantilever sheet pile walls. Now let us assume that we have got anchored bulkheads and this is the original configuration of the wall and due to some reason or due to inadequate of the section, flexure failure of the sheet pile occurs then here that plastic hinge can take place and then undergo failure. In this case tie rod anchor is in intact but the wall section is undergone failure with a deflected surface like this. So this is one of the failure modes for the anchored bulkheads which is flexural failure of sheet piling.

The other types of the failure modes are for anchored bulkheads they are basically anchor passive failure. So that means that here in this case what happened is that wall is subjected to, when it is subjected to this earth pressures this particular anchor got pulled out of this position that is because this side it has got inadequate anchorized capacity that is passive failure which are occurred that is resulted anchor passive failure. So there will be excessive movement at this point. In fact when the anchor bulkhead is there the moment at this point is zero; the deflection at this point should be as minimum as possible.

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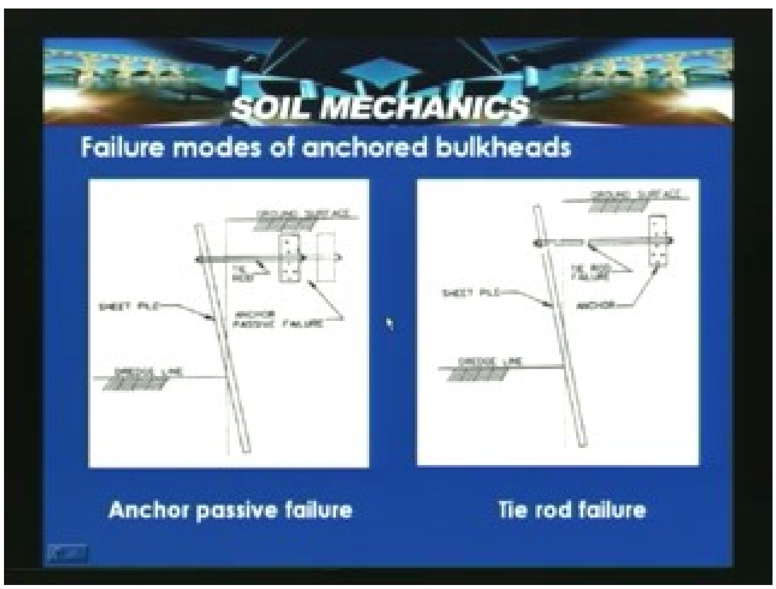

Similarly suppose if there is a chance case where a tie rod which has been kept which is having inadequate load capacity, in such situation there is a breakage failure or rupture failure of the tie rod can occur. So this is a tie rod failure which is shown here, the anchor is actually having adequate resistant but that tie rod which is actually having very low strength then tie rod failure can occur and can cause the tie rod failure. On other mode of failure is called wale system failure where the entire system is adequate but here at this point if there is a bearing plate failure occurs then we have the possibility of the failure at this point which is also called as wale system failure.

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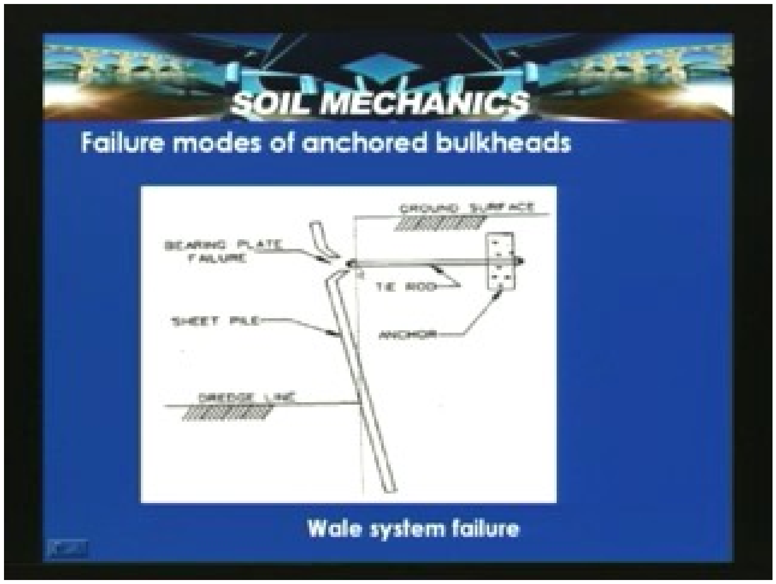

So having seen the failure modes of the cantilever sheet pile walls and anchored sheet pile walls, let us look into how analysis of this cantilever sheet pile walls and ultimately we can see how we can do the anchor sheet pile wall analysis. So in this case analysis of anchored cantilever sheet pile walls is shown, basically there is a sandy soil both sides on the active side as well as the below the ridge level side. So here entire soil medium is sand which is shown here, H is the retaining height and let us assume that wall is actually rotated about a point O which is above its tip and its located at D_0 which we will try to find out by using the earth pressure diagram and D is the embedded depth.

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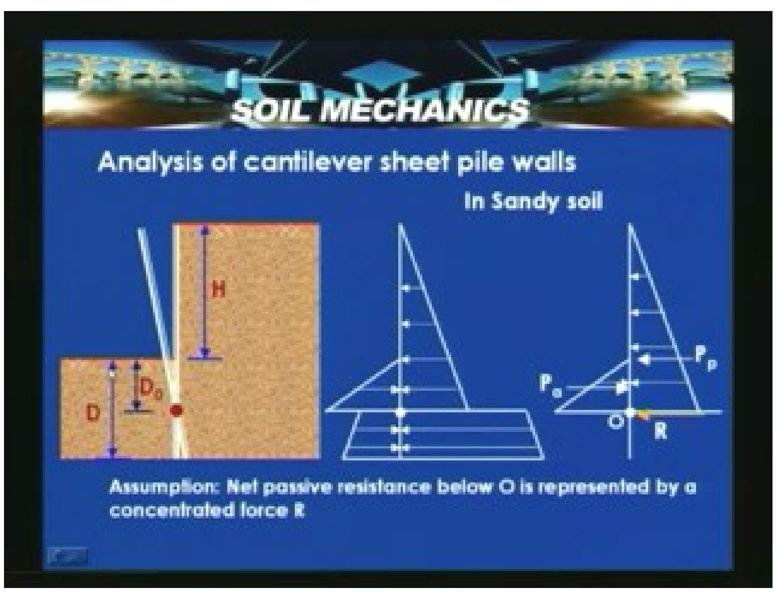

So as we discuss the earth pressure diagram will be like this, this is active and this is passive and this is again due to active case and this is due to passive case. So assumption here is that in the design of this or analysis of this cantilever sheet pile walls is that net passive resistance below point O is represented by a concentrated force R. So net passive resistance below the point O that is the point of rotation represented by a concentrated force R and that force R actually gives here is assumed to be applied just below this point O.

Now let us look in active case M_a is equal to half k_a gamma H plus D_0 whole square into H plus D_0 by 3, this is obtained if you look into the slide this is H plus D_0 so ordinate here is that k_a gamma into H plus D_0. So that is the point and this cg is acting at this point here so that's what is been given here moment of this active portion here M_a is equal to half k_a gamma into H plus D_0 whole square into H plus D_0 by 3 is the lieral (Refer Slide Time: 29:17).

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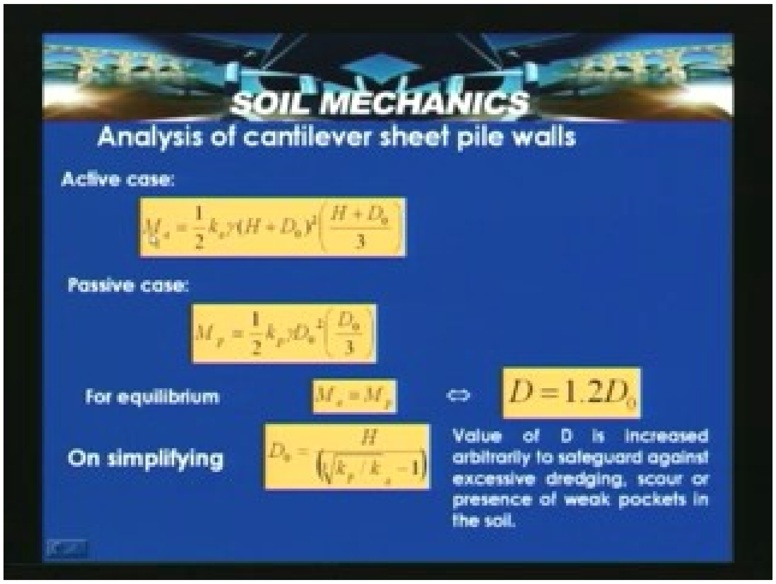

Similarly for passive case that is in front of the wall that is here this portion P_p into H plus D_0 by 3 P_a into this D_0 by 3. So M_p is equal to half k_p into gamma D_0 square into D_0 by 3. So for equilibrium conditions M_a is equal to M_p that moment about that point O has to be zero. So by equating that on simplifying we can find out that is equating these two active case and passive, moment caused by the active pressure, moment caused by the passive pressure when you equate you will obtain D_0 which is D_0 is equal to H by cube root of root over k_p by k_a minus one.

So what we generally do is that once you obtain D_0 then D is calculated by providing some 20% extra, this 20% extra is provided so in such a way that it takes care about to safeguard against the excessive dredging or scour or presence of the weak pockets in the soil.

So D is equal to 1.2 D_0 is provided, D_0 is obtained based on the deliberations which we have discussed just now. So value of D is increased arbitrarily to safeguard against excessive dredging, scour or presence of weak pockets in the soil. So value of D is increased by about 20% to safeguard against dredging or scour or presence of the weak pockets within the soil and in the further analysis of the cantilever sheet pile walls it is desirable to evaluate R by equating horizontal forces to zero and to check the net passive resistance available over the additional 20% embedded depth is equal to or greater than R. So it is desirable to evaluate R by equating horizontal forces to zero and to check the net passive resistance available over the additional 20% embedded depth is equal to or greater than R.

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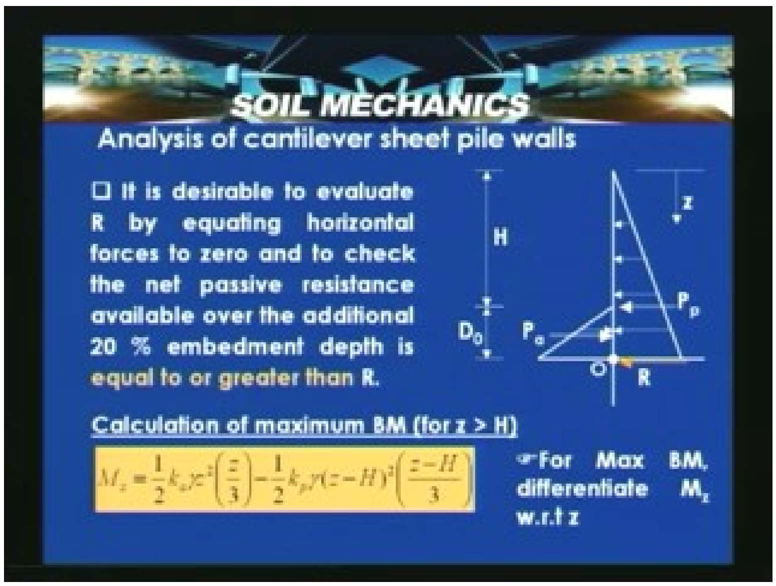

So here it is wrongly shown, this is P_p and this is P_a, calculation of the maximum bending movement can be obtained for z greater than H that is z is actually zero at this point and then is measured from this level downwards. So M_z is equal to, we can calculate half k_a gamma z square into z by 3 minus half k_p gamma into z minus H whole square that is this portion into z minus H by 3. So for maximum bending moment differentiate M_Z with respect to z we will get the location of the z where that bending movement is maximum that occurs below its level. So maximum bending moment is calculated for this simple active case for a cantilever sheet pile wall by using this method, the calculation of the maximum bending movement that is for the z value greater than H that is it maximum bending moment is assumed to occur below its dredge level.

Then in case of soil which is clay type the scenario is entirely deferent and we have discussed in the previous class that when the clay is there in the top zone which is prone for tension crack. So the tension crack zone can be occurred then we have said that the fundamental equations in earth pressures like sigma_a is equal to k_a gamma H minus 2c root k_a plus k_{aq} that is for the active case. Similarly we have got sigma_p is equal to k_p gamma H plus 2c root k_p.

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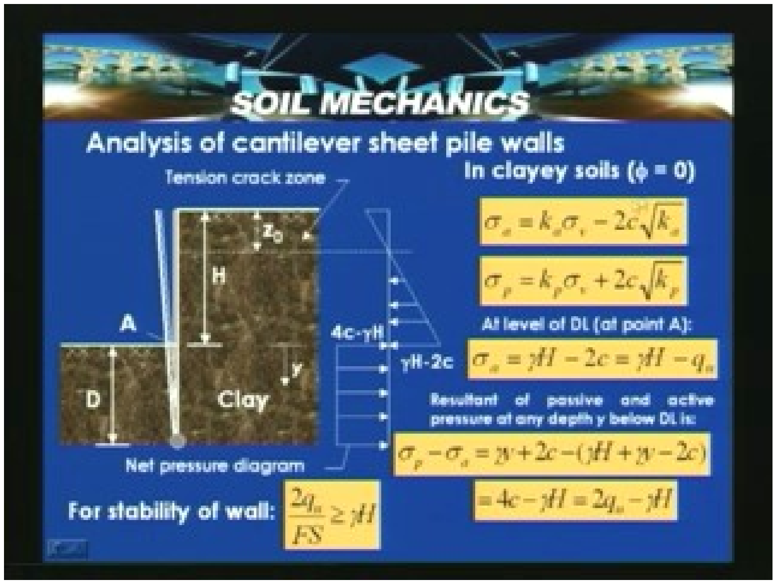

So let us apply this things, let us assume that we are having a clay soil or a soil which is cohesive in nature or the soil under un drained conditions where phi is equal to zero and again similarly in this case the wall is assumed to rotate about its point tow and this point here that D is the embedded depth and H is the retaining height. So the wall is rigid enough and undergoes a rotation about this particular point, there is also a case where wall can rotate about a certain point but which is not discussed here and in this case but mostly the rotation about this particular point occurs. So in clayey soils sigma$_a$ is equal to k$_a$ sigma$_v$ minus 2c root k$_a$ and sigma$_p$ is equal to k$_p$ sigma$_v$ plus 2c root k$_p$, at level of dredge line that is this point at point A sigma$_a$ is equal to gamma H minus 2c is equal to at this particular point that is this pressure ordinate is gamma H minus 2 c, this is gamma H minus 2 c and resultant of the passive and active pressure at any depth y below the dredge line. That is we are now trying to come here. If you look into this now in case if there is a tension crack or the soil is under tension then this particular negative pressure will occur otherwise if there is a surcharge equivalent to this pressure then the pressure will start somewhere here that will be zero at this point.

So the resultant of the passive and active pressure at any depth y below dredge level is given by sigma$_p$ minus sigma$_a$ which is obtained like gamma$_v$ that is gamma H plus 2 c minus gamma H plus gamma y minus 2c and this is the net pressure and which actually comes to be 4c minus gamma H, this 4c can be written as 2 q$_u$ because q$_u$ is unconfined compressive strength of soil minus gamma H. So now it actually evolves further stability of the wall if you look into it 2 q by FS, factor of safety should be greater than or equal to gamma H. If this condition is satisfied then we can say that the wall doesn’t require anchoring or so. If it is not satisfied then it requires anchorize at certain point below the top surface but if you look into this particular pressure, when we calculate the net pressure this is the net pressure diagram so when you look into this, this is independent of this depth and we actually we have 4 c minus gamma h which is maintained here and throughout its depth.

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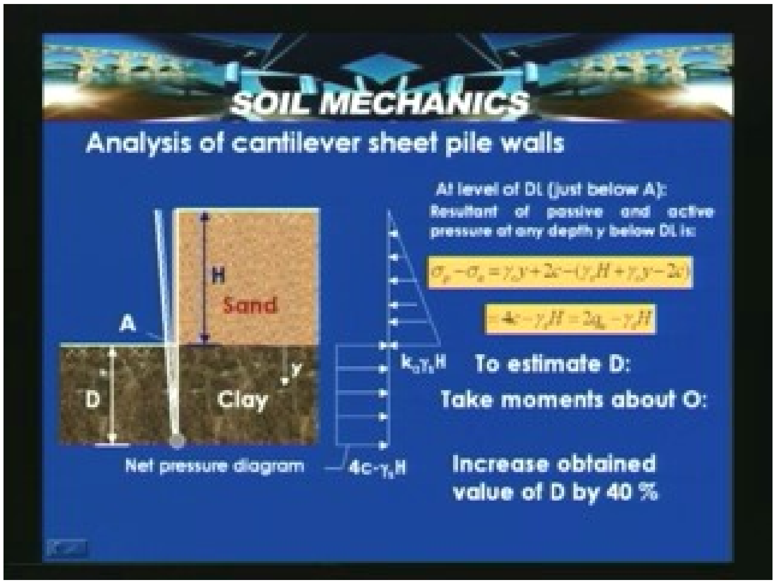

So when you wanted to calculate its net pressures which are acting or moment about this particular point then we can actually calculate by taking this forces and then taking moments about this particular point. So in the analysis of cantilever sheet pile walls in clay soils, using the fundamentals what we discuss and applying the relevant earth pressure coefficients. If it is layered soils then we need to apply the relevant coefficients and we can calculate the earth pressures and arrive at the net pressure diagrams and then calculate the required features and check the required depth whether it is adequate or not and in this case for example this as a thumb rule for the stability of the wall two cube by FS if you are able to maintain greater than or equal to gamma H then actually we can say that the wall is stable.

Let us assume that a case where you have got a sandy soil at the top and clayey soil at the bottom and then wall is rotating about its tip. So at dredge level that is at this point just below point A, so if you look into this here we have got two types of soil, one is sand, one is clay. So resultant of the passive and active pressure at any depth y below the dredge level is sigma$_p$ minus sigma$_a$ again gamma$_c$ y that is gamma$_c$ y plus 2c that is this point minus where gamma$_s$ h is that over burden (Refer Slide Time: 37:18) in to this and k_a is now applied below the wall. So k_a is equal to 1 for phi is equal to zero soil that is this is the soil which is clay having phi is equal to zero so k is equal to1, that factor is not taken because k_a is equal to one here and k_a gamma$_s$ h gamma$_c$ y that is this portion minus 2c. So when we look into this, when you simplify 4 c minus gamma$_s$ h it yields 2q_u minus gamma$_s$ h where gamma$_s$ is the unit weight of the sand. To estimate D take moments about point O. Once we get this the net pressure diagram is k_a gamma$_s$ h that is due to sand and 4c minus gamma$_s$ h. So to estimate D take moments about O and increase the obtained D by 40% to again safeguard against excessive dredging or scour or due to any weak pockets in the soil. So this is the case of analysis of cantilever sheet pile walls and in case if you have a sandy and a clayey soil.

Now let us look into the feature like how we can design anchored bulkheads and we said that these anchored bulkheads are possible when you have got wall heights more than 5 meters or 6 meters also. In such situations this anchor bulkheads can be provided with a tie rod and a placement of the dead man anchor.

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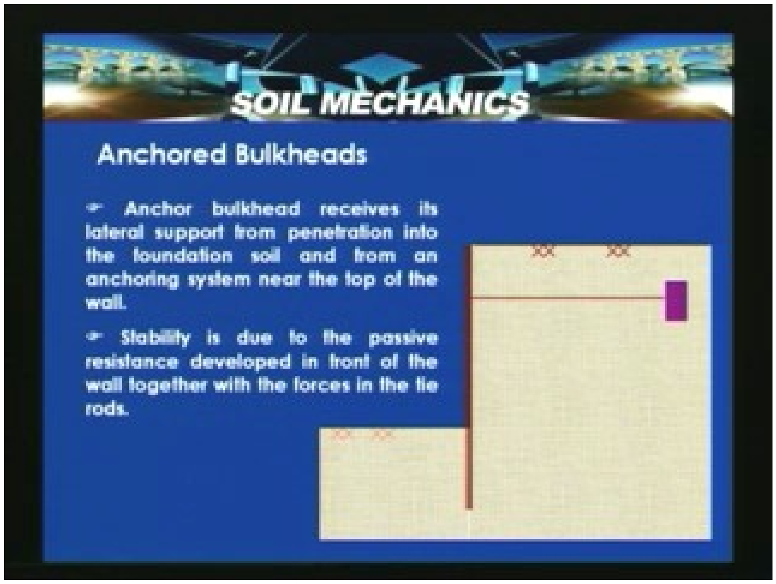

So in the forth coming slides what we look into is that how these anchored bulkheads can be analyzed and how we can calculate the forces in the tie rod or it is also in practice to use some advanced programs like finite element method, other programs to compute and design these walls. So anchored bulkhead receives its lateral support from penetration into the foundation soil and from an anchoring system near the top of the wall. So this anchoring system will be at the top of the wall sometimes these are called anchored sheet pile walls and sometime the anchors also can be at 45 degrees they are called ground anchors. So the sheet pile walls are provided either with these types of systems or with ground anchoring system. Stability is due to the passive resistance developed in front of the wall together with the forces in the tie rod.

So when the wall previously a cantilever sheet pile wall is without any tie rod here or anchoring system but in order to manage the deeper excavation particularly especially in in water front structures along the melon environment, to ensure the stability a passive resistance is developed in front of the wall together with the forces in the tie rods. So these tie rods have to be placed at certain spacing and then should have sufficient length. So that they will not interference with the failure surfaces. Basically these anchored bulkheads are analyzed with two methods and two method of analysis, one is called free earth method. In this case a free earth method is used, if the depth of embedment depth is less than 0.2 to 0.3 times the height, H is the height of the retained height which is actually this height which is shown here, H is the retained height and D is the embedded depth here.

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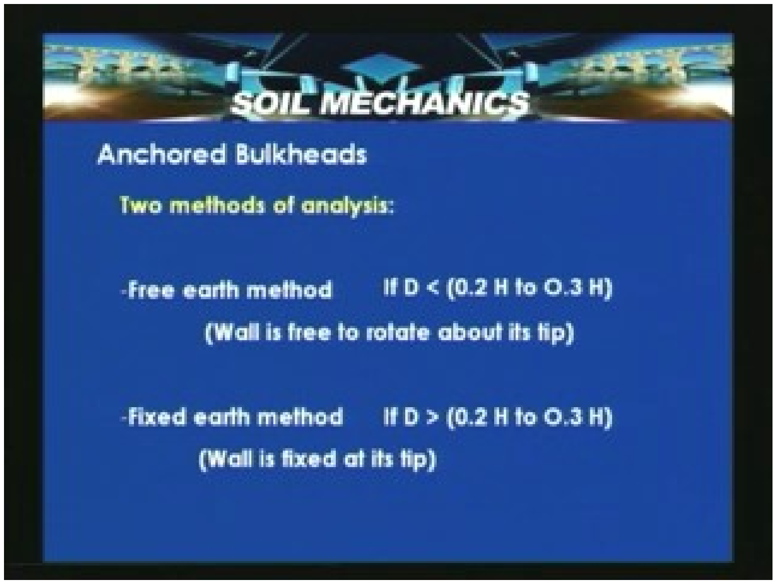

Now free earth method that is if the D is less than 0.2 to 0.3 times the H, the free earth method is used. In the free earth method it is assumed that the wall is free to rotate about its tip. In the fixed earth method if D is greater than 0.2 H or to 0.3 H. Generally the fixed earth method is adopted that is this is fixed in such a way that the wall is embedded with deeper depths so it ensures the wall fixity. So fixed earth method is generally used if the depth of the embedment is large that is more than 0.2, 0.3 times the H and wall is fixed at its tip firmly. So in the anchored bulk heads out of these free earth method is frequently used in practice here what we knew is that this is the wall section and this is that anchor rod or a tie rod force acting away from the wall movement.

So if this is the wall movement direction this acts in the opposite direction and this is that active pressure diagram and this is the passive pressure diagram and this is here the active thrust will act occur and here the passive thrust will act and this is the point A that is the point where the anchorize has been done with a sheet pile wall section and H is the retaining height and D is the embedded depth. So here the depth of the fixity is assumed to be inadequate to produce fixity at the lower end of the wall.

So in this free earth method what is assumed is that the depth of the fixity is assumed to be inadequate to produce fixity at the lower end of the wall and here in this case because of the movement there is p_a this is the deflection line, this deflection line which is shown here and here there is no movement and there is movement towards this side because of this location of the anchor, the wall is pulled inside and there is a movement here. So the depth of the fixity is assumed to be inadequate, this condition can occur if there is no anchor passive failure. So the depth of the fixity is assumed to be inadequate to produce fixity at the lower end of the wall.

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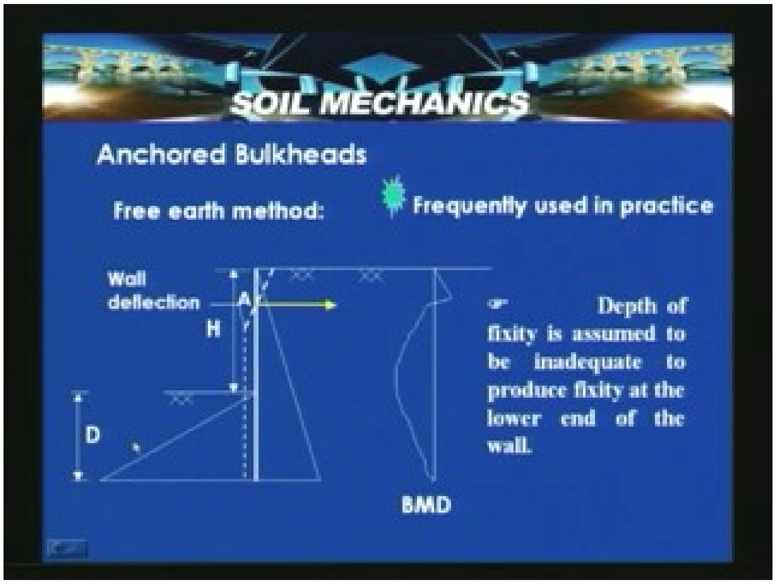

Then in case of the fixed earth method the sheet pile walls are driven to a considerable depth to ensure fixity or a firm fixity to the sheet pile walls. So here the D is large compared to the previous case and so that there is no movement at this point and this is the deflection line in this case, you can see that the difference in the deflection line and this is the active pressure diagram and this is the passive pressure diagram and this is the anchor force which is acting and here what you see is that this is the bending movement diagram which is shown and this is the point of contrafricture (Refer Slide Time: 43:43) where the bending moment is zero.

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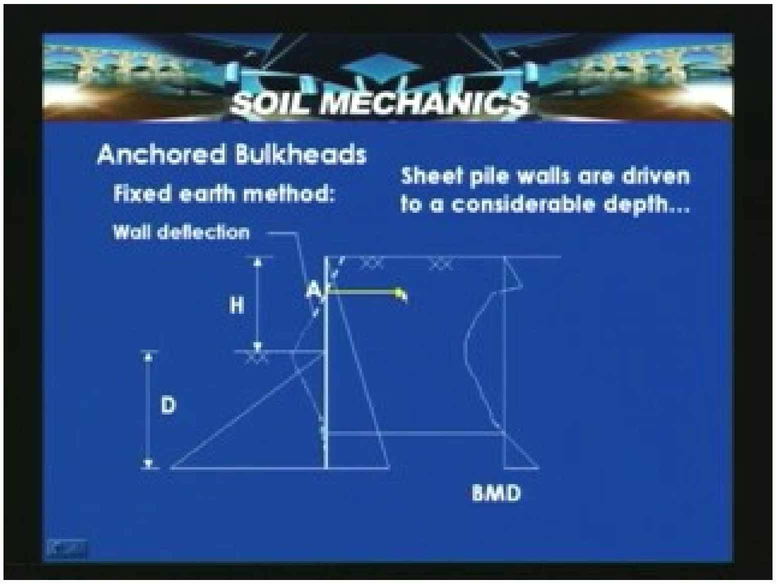

So in the fixed earth method the deflection is shown here like this and there is no movement of the wall because of ensuring or a sheet pile by ensuring that sheet pile walls are driven to a considerable depth. So in the free earth method let us look how we can design and we are not discussing in detail about the fixed earth method but the details can be obtained from any soil mechanics dispose. So in the free earth method they assume D that is the depth of the embedment in front of the wall and calculate the active and passive earth pressures.

So second step is to calculate active and passive pressures and if you are having a layered soil consider and if you are having say presence of water pressure that is the differential water has to be considered in the net pressure diagram. So required depth of the embedment depth D can be calculated or determined from the condition that moment about the anchor point must be zero. That means you need to take moments at this particular point and ensure the moment is zero that is moment of this active thrust about this particular point is taken and ensure that is calculated and determined depth of D. So the required depth of the embedment depth D can be calculated from the condition that sigma M_a must be zero this condition yields a cubical equation and which can be solved by substituting trial values.

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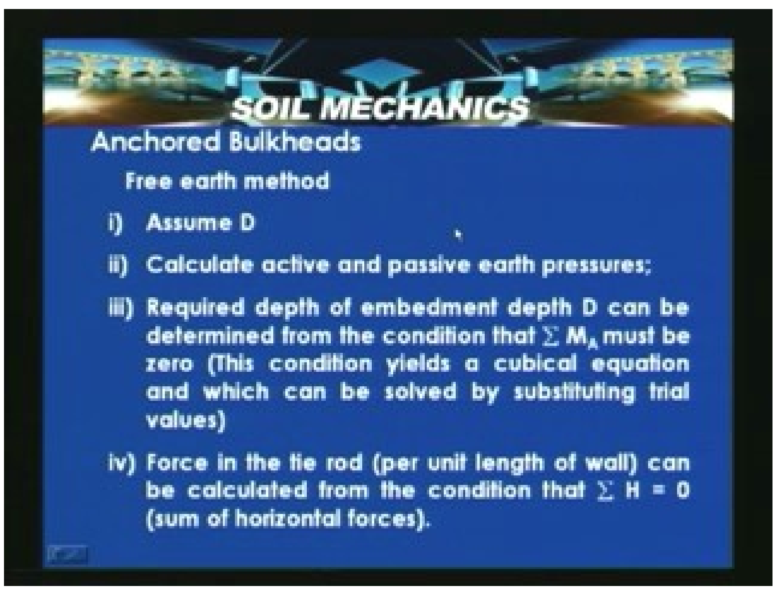

So this is the case where you have got sandy soil on both sides so in this case you will be getting cubical equation but if this having a clay then the pressure diagrams will be same as what we discuss like 4 c minus gamma H net pressure in front of the wall into the entire depth. So it yields and it can determine the pressure moments about point A and forces in the tie rod per unit length of the wall can be determined by seeing that the summation of the horizontal forces that is P_a and P_p with that we will be able to calculate the forces in the tie rod. It need to calculate whatever may be the passive forces whatever may be the active forces the net difference has to be compensated by this anchor rods or tie rods.

Also one important thing as we discuss the location of the anchor plate is very important. So if this is the wall cross section and if this is the tie rod and this is let us say that this is a plate which is located having a depth HA anchor plate and DA is the depth that is the tip of this anchor plate from the ground surface, LA is this length of this anchor plate from here that is length of this location of the anchor plate from the inner face of the wall. Now in this case, this particular zone is active zone and this particular zone is passive zone and this particular zone is again passive zone and this particular zone is active.

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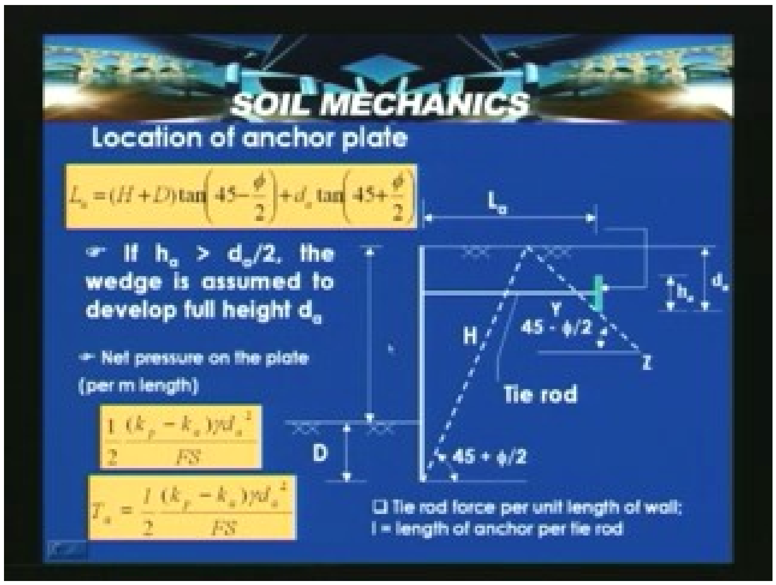

So now the location of the anchor plate or L_a can be determined like this. This is from the 45 plus 5 by 2, if it is considered then these are nothing but length of this portion and this portion so this portion which is H plus D tan 45 minus phi by 2 plus d_a tan 45 plus phi by 2 when you put this then you will get that entire length that is called L_a. So we have to ensure that it is less than this, suppose if it is within this point then there is a possibility that it will not have any ridicuvate anchorize (Refer Slide Time: 47:37) capacity that means it has to be seen that it is beyond this particular line which is shown here in this case. That is passing through this strip and beyond this plane passing through z, if h_a that is this point is greater than d_a by 2, the wedge is assumed to develop full height d_a that is if h_a is greater than d_a by 2 then we will assume that the full height is actually participating but this is the actually common case if h_a is greater than d_a by 2 the wedge is assumed to be develop full height.

Now the net pressure on the plate, this particular plate per meter length is given as this is we know that passive and this is active. So in case active diagram and this is a passive diagram will come, when you compute and apply a factor of safety and d_a is the depth. So because we assumed that h_a is greater than d_a by 2 so the entire full wedge is participating so half k_p minus k_a into gamma d_a square by a factor of safety which is shown here.

So now the tie rod force per unit length of the wall can be determined by using this expression T_a is equal to half k_p minus k_a into gamma d_a square by factor of safety that is l is this length of the anchor plate per tie rod that is the l, so by using this procedure what we have discussed is that first we have to design the value d and increase again by 20% to safeguard against scour and other reasons which we discuss. Then calculate the tie rod force and design the tie rod force and design the section which is required for providing based on the z and also locate the section safely. So that we will have an adequate anchorize capacity and also ensure that a good compaction for a fill. So that in this zone when there is post installation in this particular zone if adequate compaction of a material is ensured its possible that it can have a better anchorize capacity.

So in this earth pressure lectures what we try to understand is how to calculate earth pressures and based on this earth pressures we said that how this earth pressures can be used for the design of the earth retaining structures and we discussed about typically two theories, one is Rankine theory another one is coulomb's earth pressure theory and then we discussed about some example problems in understanding how we can calculate earth pressures for the different conditions.