Foundation Engineering
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Module - 02
Lecture - 08
Pile Foundations - 3

In the last two lectures, related to Pile Foundation, we saw various aspects related to construction of piles, it is type of piles. And then, in the last class, we started with that, how you can estimate the load carrying capacity of pile, under compressive load. There were four categories; that we divided various methods into and they were static pile load formulae, then, pile load test, dynamic pile load formulae and then correlation with penetration data.

We started with static pile load formulae and now let us see that, how we can put in mathematical form or how we can work out the load carrying capacity of pile under compressive load using a static load pile load formulae. You know that, there are two type of resistance, which will be mobilized as the pile is subjected to the compressive load as soon as the compressive load is applied, what happens is that, the first load gets mobilize, along the pile shaft to some length of the pile.

And as you go on increasing, that applied load the load gets mobilize along the length full length of the pile shaft. Once, that particular load becomes the equal to ultimate skin friction resistance and then it starts resulting in the point bearing resistance.

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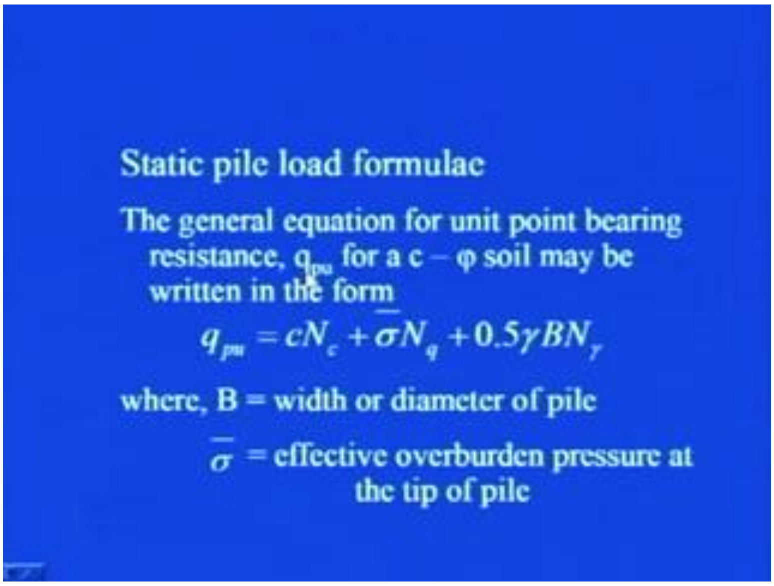

So, the general equation for unit point bearing resistance, that is q p u for a c phi soil, may be written in this form, which is q p u is c N c plus sigma bar N q plus 0.5 gamma BN gamma, where this term B is width or diameter of the pile. As I told you, that the pile cross section can be circular, can be square, can be rectangular. So, in case of a square pile, this B is the diameter of the pile and in case of square or rectangle, it is the width of the pile.

Then, sigma bar is the effective overburden pressure at the tip of the pile, since we are talking of this point bearing resistance, which is unit point bearing resistance. So, that will be occurring at the tip of the pile; that is why this sigma bar is effective overburden pressure at the tip of the pile.

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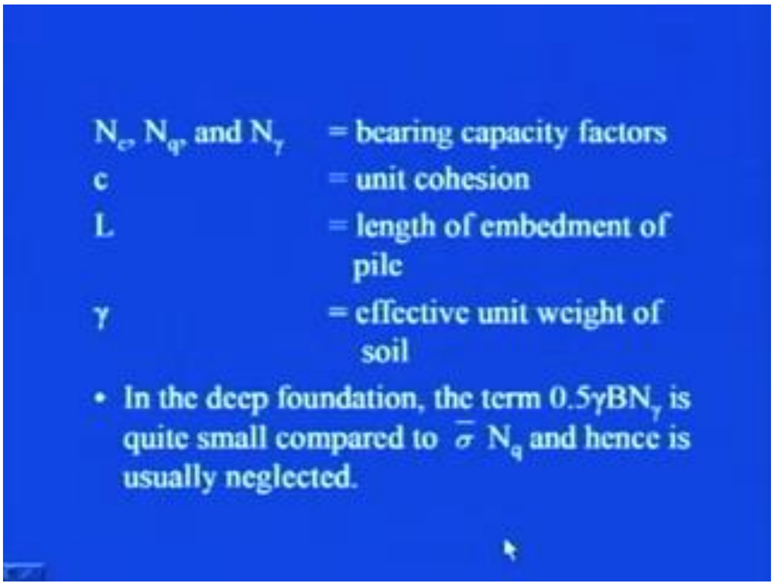

Then, as you have studied in the chapter of shallow foundations that N c, N q and N gamma, they are bearing capacity factors, which can be determined based on different theories given by different research workers. C is unit cohesion; L is length of embedment of pile; that is whatever length of pile is being surrounded by the soil and gamma is effective unit weight of soil.

In the deep foundation, the term B 0.5 gamma B N gammas, that is this term, you see there are three terms c N c sigma bar N q plus 0.5 b gamma B N gamma. So, what happens in case of deep foundation, this last term that is the third term 0.5 gamma B N gamma is quite small as compared to sigma bar N q and hence it is usually neglected.

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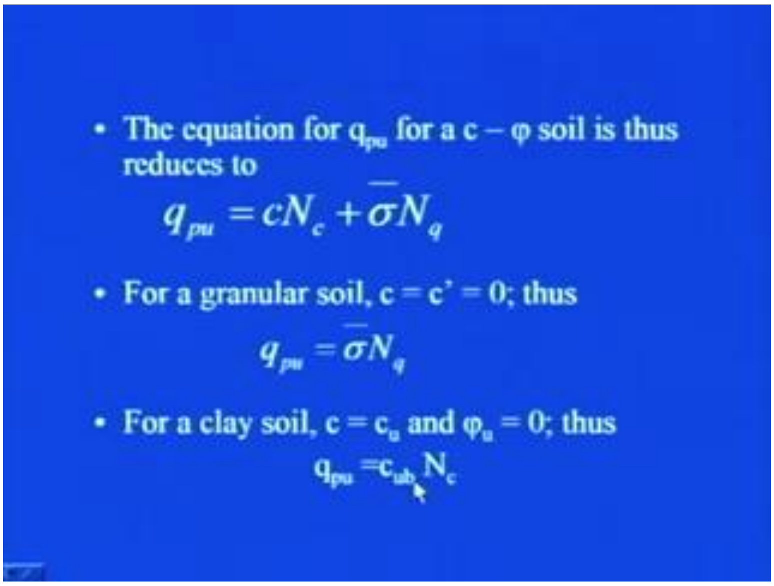

So, the resulting equation for unit point bearing resistance for a c phi soil gets reduced to only two terms, that is c N c plus sigma bar N q, where N c and N q are bearing capacity factors c is unit cohesion, sigma bar is effective overburden pressure at the tip of pile. Now, for a granular soil, this one is the general equation for any type of soil; that is c phi soil.

In case, the soil is purely granular soil; that is c is equal to 0 or may be this is unit cohesion or effective one, they both will become 0. And thus, this in this equation the first term will become 0 and the only remaining term will be as q p u is equal to sigma bar N q. For a clay soil, what will happen, in undrained condition your angle of internal friction will become 0 and c will work out to be undrained cohesion.

So, this second term which is there will go and because this N q value, it depends on the value of phi and when it is 0, this will become 0. So, q p u will simply be equal to c N c and c is equal to c u, so this will be c u b, b stands for base; that is at the base of the pile, at the tip of the pile. So, c u b into N c and N c is bearing capacity factor c u b is undrained cohesion at the base of pile or at the tip of the pile.

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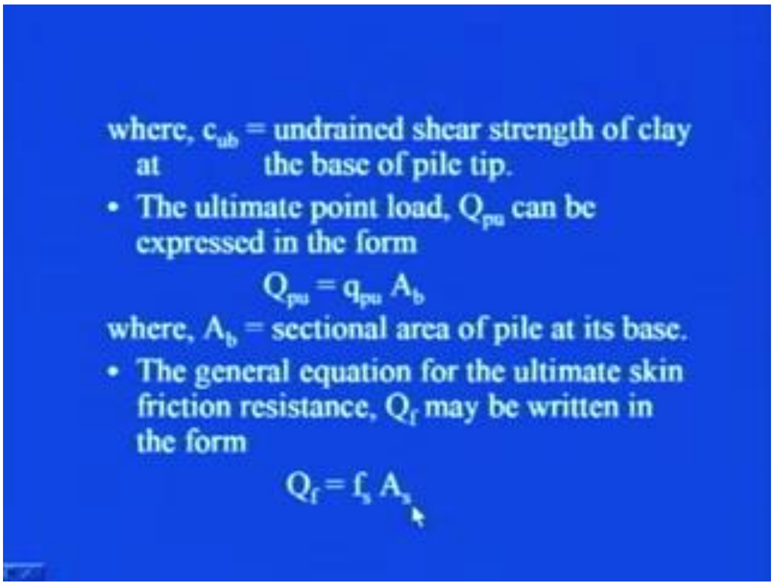

You see here c u b is undrained shear strength of clay at the base of pile tip, then the ultimate point load that is Q p u, see earlier we were talking of unit point bearing, that was the pressure. Now, if I we talk in terms of load, you have to multiply that intensity with the area on which it is acting. So, when you multiply this q p u, that is the intensity by the base area you will get ultimate point load which is resisted or which is due to the area of the pile tip.

So, it can be expressed in the form, that Q p u is equal to this small q p u into A b, where A b is the sectional area of the pile at it is base. In case, the pile is circular with diameter d, then this A b will be equal to phi by 4 d squared and in case if it is say square of side b, then in that case this A b will be equal to b square. So, what is all about, this point ultimate point load.

Now, let us see that how this ultimate skin friction can be evaluated, so the general equation for the ultimate skin friction Q f, may be written as that is Q f is equal to f s into A s. You have seen that, here also we multiplied this intensity by the area on which it was acting and this multiplication resulted into the ultimate point load. So, here if you want to find out ultimate skin friction resistance q f, we will be multiplying this unit friction by the area on which that it will be acting.

Now, what is that area on which this skin friction is getting mobilized, as we have seen in the last class; that the skin friction gets mobilize along the pile shaft. So, the surface area of the pile shaft is the area on which this f s; that is unit skin friction resistance acts.

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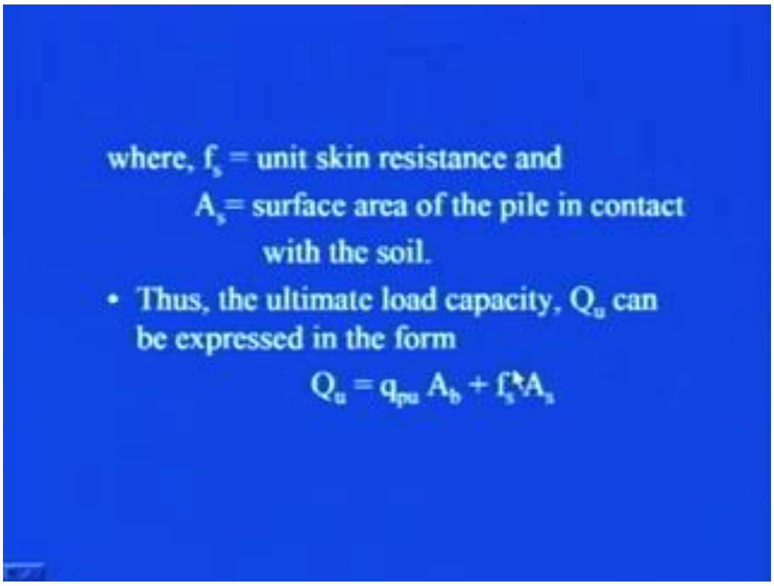

So, f s is your unit skin friction resistance, A s is surface area of the pile in contact with the soil, because when the load is coming to the pile, there is vertical movement. And because of that, shear stresses are getting generated and they are resulting into skin friction resistance. So, along this surface area of the pile, which is in contact with the soil, this unit friction resistance will get developed and if you multiply these two, you will be getting the ultimate skin friction resistance.

And as we discussed in the last class, that the total ultimate load carrying capacity of the pile will be the total skin friction resistance plus total point bearing resistance. So, here you can see, we can write it in this expression form, that the ultimate load capacity Q u can be expressed in the form Q u is equal to q p u into A b plus f s into A s. Where, this q p u is the unit point bearing acting on the area of the base of the pile, that is A b multiply them, you will get the total point bearing load.

And then, this f s is unit skin friction acting on the surface area of the pile shaft, you multiply them and you will be getting the total skin friction resistance.

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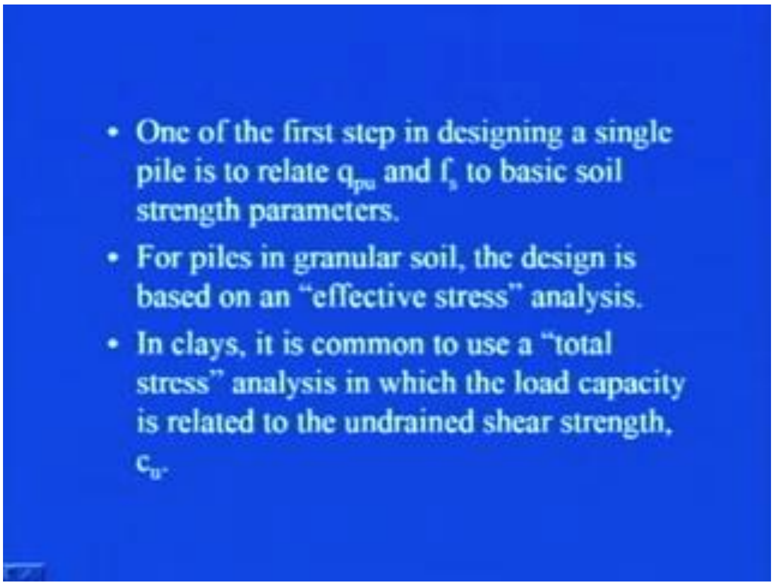

So, there are few points; that we must keep in mind, so let us try to see one by one that what are they, so one of the first step in designing a single pile is to relate q p u and f s to basic soil strength parameters. See, basic soil strength parameters are what they are c and phi shear strength parameters, that is cohesion and angle of internal friction. Depending on what type of soil is being surrounded, that is what type of soil is present surrounding the pile.

The distribution that this much of the load will be shared by skin friction or this much of the load will be shared by point bearing, they will be decided upon. So, we have to relate first of all that unit point bearing and unit skin friction to basic soil strength parameters, for piles in granular soil, the design is based on an effective stress analysis. So, let us say that the pile is being driven or bored in granular soil; in that case c is equal to 0; that is cohesion will be 0.

So, in that case the design is based on effective stress analysis; that means that all the stresses that you will be taking, let us say overburden stress, so that all you will be taking as effective one. In clays, it is common to use a total stress analysis in which the load capacity is related to undrained shear strength. So, you see what exactly is the difference between the granular soil and the analysis in clays.

Now, let us start with the piles in granular soils, granular soils means, that if a pile is driven into sand and gravel, so first driven piles.

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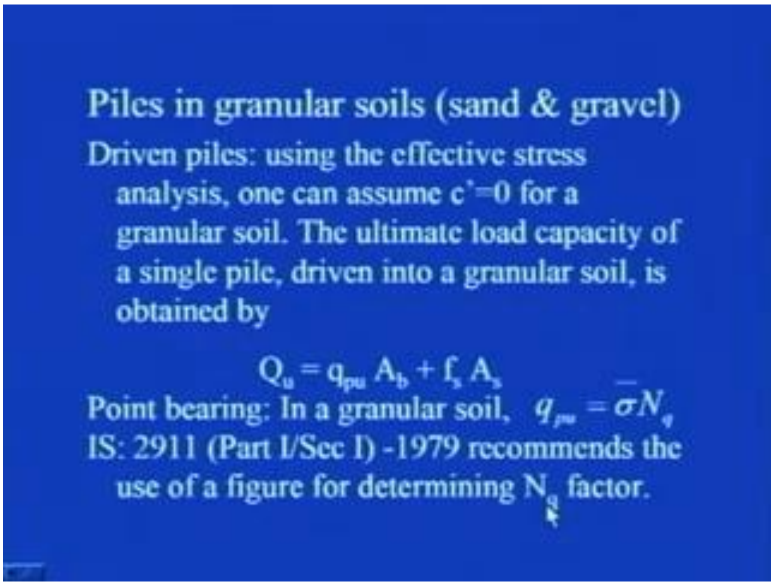

Using the effective stress analysis, one can assume c prime that is effective cohesion to be equal to 0 for granular soil. So, the ultimate load capacity of a single pile driven into the granular soil can be obtained by using this general expression, that is Q u is equal to q p u, A b plus f s into A s. So, point bearing, first let us try to see that how we can find out this point bearing, this q p u; that is point bearing unit resistance.

That in a granular soil, it is q p u is equal to, as we know that we had general expression in which three terms were there and then the last term which we said that it is very small as compared to the other two terms. So, we ignored that and then we left with two terms, that is c N c plus sigma bar N q and here in case of granular soil your c will be equal to 0, so we will be left with only one term, which is equal to sigma bar N q.

Now, how you can find out this term N q, as you have studied in case of shallow foundation depending on the value of angle of internal friction, you can pick the value of this N q by standard charts which are available depending on various analysis, which is available. But, I am just giving you a feel of that that, what exactly our IS code recommends as far as this N q values are concerned.

So, IS: 2911 part I, section 1979 recommends the use of a figure for determining N q factor.

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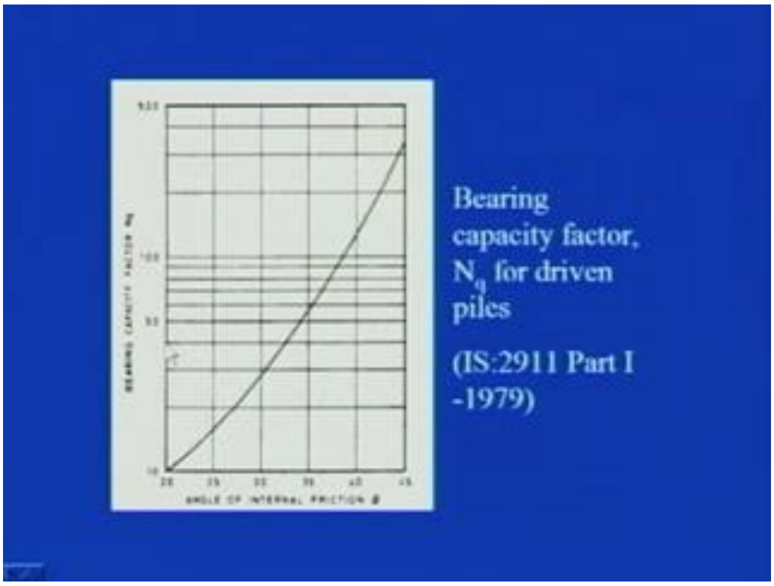

You can see here, that here is the angle of internal friction and here is bearing capacity factor N q and then this curve has been drawn. So, depending on the value of angle of internal friction which you will know as it is a property of the soil which is surrounding the pile. Let us say, that you have angle of internal friction value to be equal to 30 degree.

So, you go to x axis pick a value is equal to 30 degree, then you move along this vertical line, wherever that cuts, this particular curve. You see, this is the curve, you move along this 30 line, wherever it cuts here, you read the corresponding value of N q and that value you can use safely for all the computation process. Let us say that, there is some value between 30 to 35, that is let us say 32, so you approximately read, where this 32 will lie on this particular axis go here read the corresponding N q value.

As you can see that here, it is on normal scale, but here it is on log scale, so this is 10, 20 30, 40, say 50, so you see here if you take some value let us say roughly in between 30 to 35 that is 32.5. So, you can see here that this value is this value of N q is approximately 40, so this is how you can find out the value of this bearing capacity factor. However, there are various other methods also available, like Terzaghi's bearing capacity factors, Meyerhof's and all other things, but usually this we adopt from IS code method.

Then, for a driven pile in sand the value of phi is equal to phi 1 plus 40 degree by 2, may be taken, where phi 1 is the in-situ value of angle of shearing resistance.

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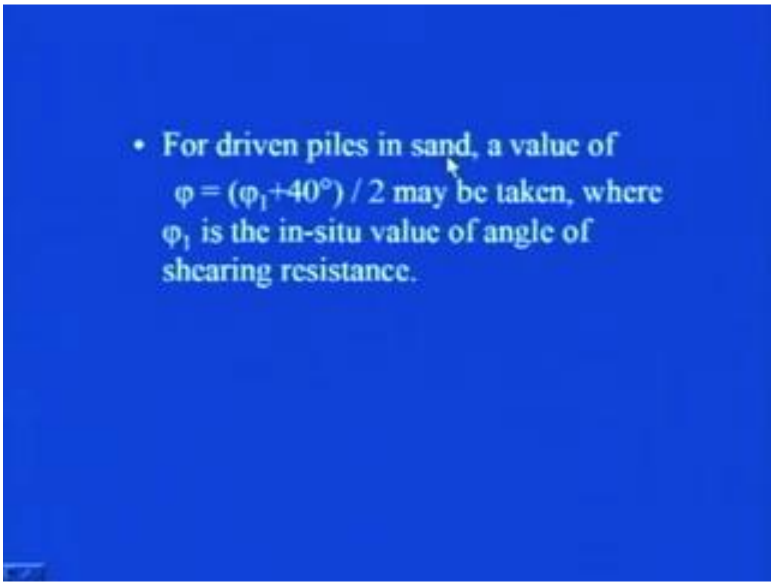

So, you see here, if your shear strength parameters, they are c and phi 1, that is their in-situ shear strength parameter, you have to modify the value of phi as per this particular expression, in case you are driving the pile in cohesion less soil, that is in case of sand. So, for driven piles in sand, instead of taking phi is equal to phi 1, which is in-situ angle of internal friction, you simply modify that value as phi 1 plus 40 degree by 2.

And then, whatever is the phi, which you get, you can go to the previous chart as I showed you in case of from IS code and then you can read that corresponding value of N q. Thus, with known pile dimension and soil properties, the ultimate load capacity Q p u can be determined.

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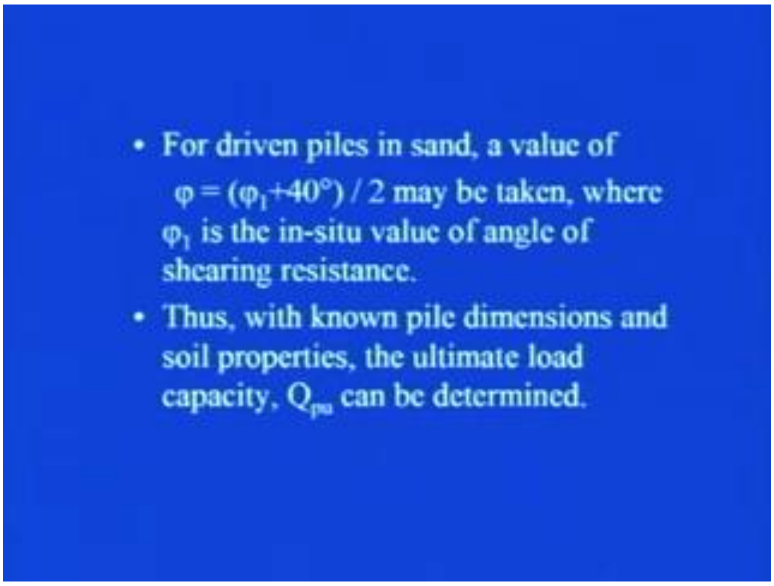

You see, in case of granular soil, that small q p u is equal to your sigma bar into N q, so N q you are getting knowing this property shear strength parameter of the soil, you know that sigma bar, what exactly will be the effective stress at the pile tip. And then, if you simply multiply that by area of the base of pile tip, you will be getting this q p u which is the ultimate load capacity in point bearing.

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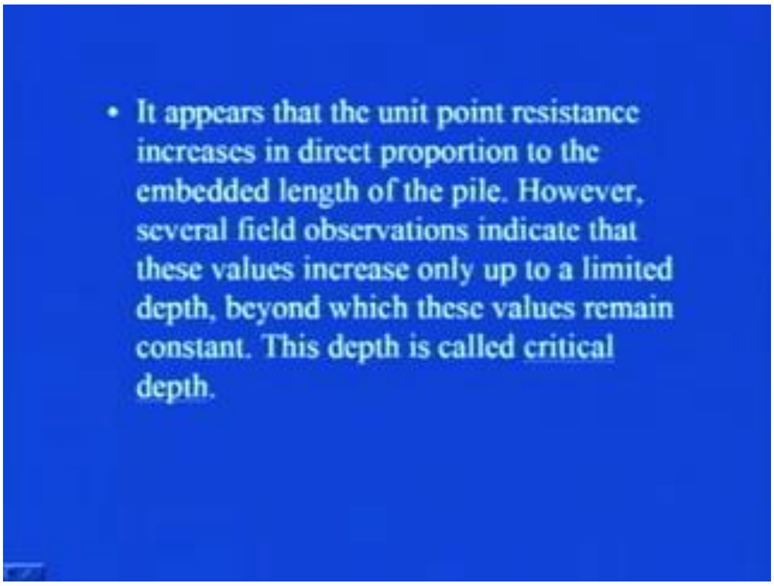

It appears, that the unit point resistance increases in direct proportion to the embedded length of the pile, because you see that effective stress, that is sigma bar into N q is your unit point resistance. In the absence of any water table, what will happen your sigma prime will become equal to the total stress and that total stress we can write down as gamma into z.

So, what happens, where the ground surface is there where z is equal to 0, that sigma will become 0 and as you will go on coming towards the tip of the pile the z will be increasing and so you expect a linear increase in unit point resistance also. However, several field observations indicate that these values increase only up to a limited depth, beyond which these values remain constant and this particular depth is called the critical depth.

So, what happens is gamma times z, this value will go we expect that this will go on increasing as you go deeper and deeper from the ground surface. But, what happens practically is that, this value increases up to a certain depth and beyond that, it just becomes constant. So, that particular depth till which it increases linearly that is called critical depth.

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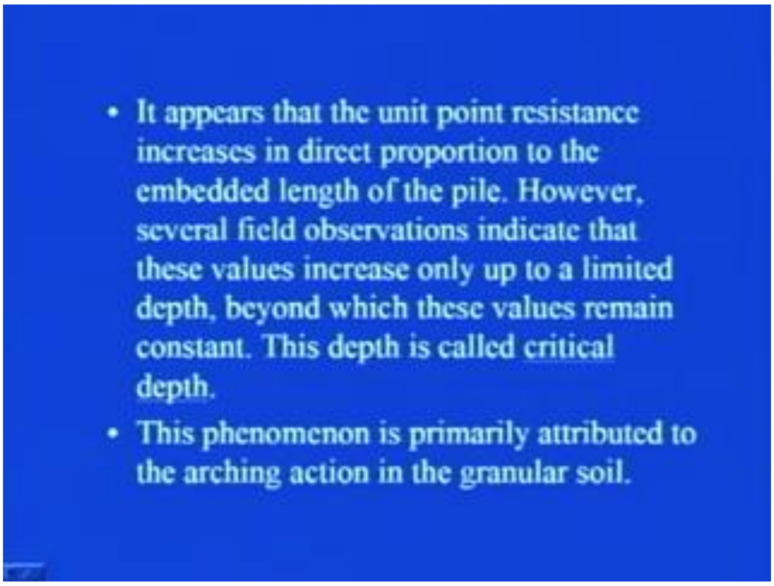

This phenomenon is at primarily attributed to the arching action in the granular soil; see this we are talking in terms of driven piles in granular soil. So, you have to be very careful using this critical depth concept, in case of granular soil only you will be using, because arching action of the soil is present in granular soil only.

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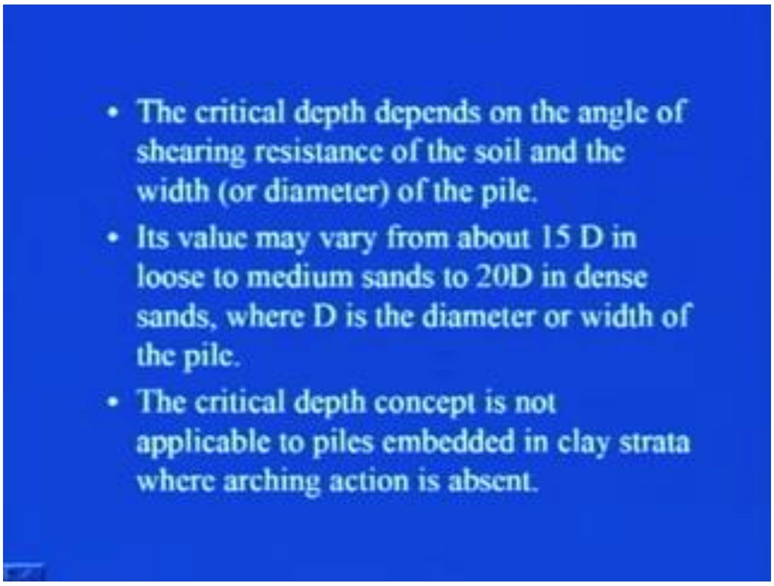

Now, this critical depth depends on the angle of shearing resistance of the soil and the width or diameter of the pile, depending on what exactly is the width or depth diameter of the pile and what is the shear strength parameter of the surrounding soil, this value of critical depth will be changing from one particular pile to another particular pile type. It is value may vary from about 15 times diameter in loose to medium sands to 20 times diameter in dense sand, where diameter where d is the diameter or width of the pile.

So, you see here, in case if the pile is surrounded by loose to medium sand, roughly you can take this critical depth value to be equal to 15 times the diameter of the pile. So, if the diameter of pile is say one meter, in case of loose to medium sand, the critical depth can be approximately equal to 15 meter and in case of dense sand it can be of the order of 20 meters. This critical depth concept is not applicable to piles embedded in clay strata, where the arching action is absent.

So, while you go for soil exploration, you will be know that what exactly is the soil strata below the ground surface and accordingly, because all these things you will be doing before hand, before going for the construction of the pile. To get an idea, what exactly or what can be approximately the load capacity of the pile under compression.

So, in case you know, that if the soil strata is granular in nature, you will use this critical depth concept and in case if the clay strata is present or the clay is surrounding the pile, then in that case this critical depth concept will not be applicable. In that case, you have to consider that whatever is the linear variation in which the unit point resistance will be increasing, that only you have to take into account.

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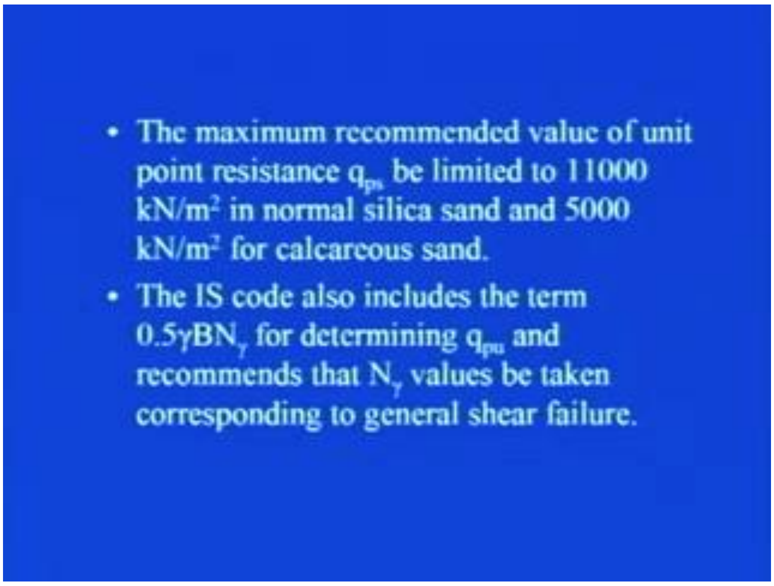

So, the maximum recommended value of unit point resistance, that is q p s is limited to 11,000 kilo Newton per meter square in normal silica sand and 5,000 kilo Newton per meter square for calcareous sand. The IS code also includes the term 0.5 gamma B n gamma, for determining Q p u and recommends that N gamma value be taken corresponding to general shear failure.

Although, I told you that the last term which was present here in the most general equation of q p u, that we can neglect it. But, the IS code includes this term and then N gamma value, it also recommends that N gamma value be taken corresponding to general shear failure.

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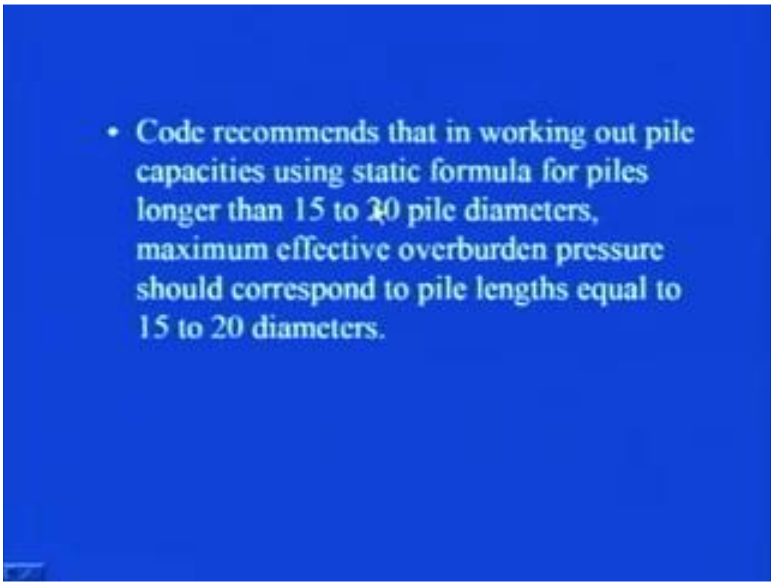

Then, code recommends that in working out pile capacities using static formula for piles longer than 15 to 20 times the pile diameter, the effective overburden pressure should correspond to pile lengths equal to 15 to 20 diameters. So, you see here, that let us say that the pile length is equal to say 30 meter or even more than that, but what code recommends is that, when you find out this pile capacity using static formula for longer than 15 to 20 times pile diameter.

That is, if the pile diameter is one meter and the length of the pile is more than this 15 to 20 meters, say if pile diameter is equal to one meter, then 15 to 20 pile diameters will become 15 to 20 meters. And if, the length of the pile is more than this, that is 15 to 20 meters, let us say 30 meters in this case, so how you will find out this effective overburden pressure. That you must take equal to 15 to 20 times the diameter, that is in this case 15 to 20 meters.

Even though, the pile length is 30 meters let us say, but then you will be considering the maximum effective overburden pressure to only 15 to 20 meters, that is the recommendation from IS code.

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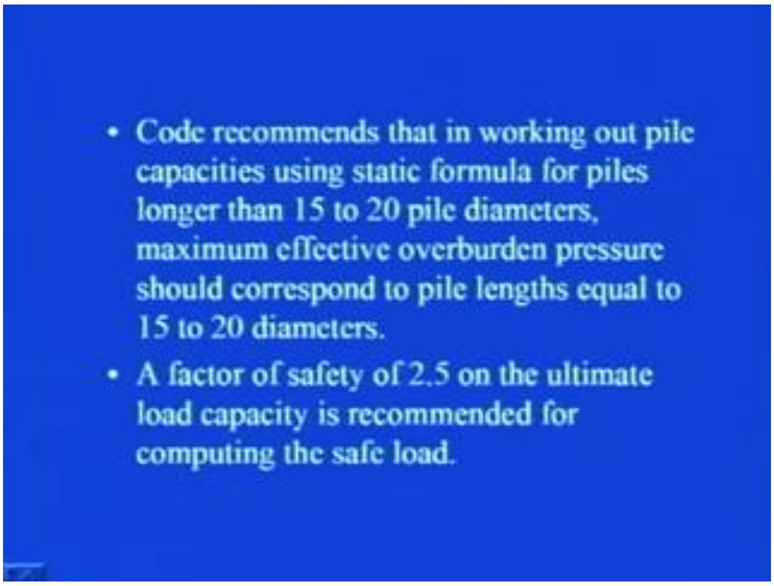

A factor of safety of 2.5 on the ultimate load capacity is recommended for computing the safe load. So, whatever is the ultimate load capacity; that you get simply divide that by this factor of safety of 2.5, to get the safe load on the pile, that the pile can take.

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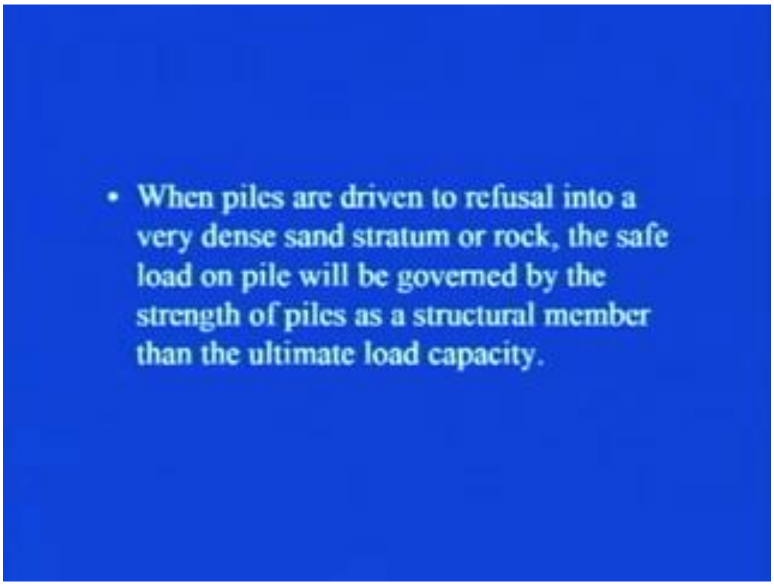

When, piles are driven to refusal into a very dense sand stratum or rock, the safe load on pile will be governed by the strength of piles as a structural member than the ultimate load capacity. See from here, you are finding out the ultimate load capacity, but the pile is made up of let us say steel or concrete. So, in case the pile is driven to refusal to very dense sand stratum or rock, in that case the governing factor will become the strength of the pile as a structural member as compared to the ultimate load capacity.

So, in that case safe load you have to take as the strength of the pile as structural member not the safe load, which you are getting through ultimate load capacity, this we were talking related to the point resistance. Now, let us try to see that, how this skin friction can be determined or estimated in case of piles in granular soils. So, the ultimate skin friction resistance, which I am representing by Q f is obtained from Q f is equal to f s into A s, after determining the unit skin friction f s.

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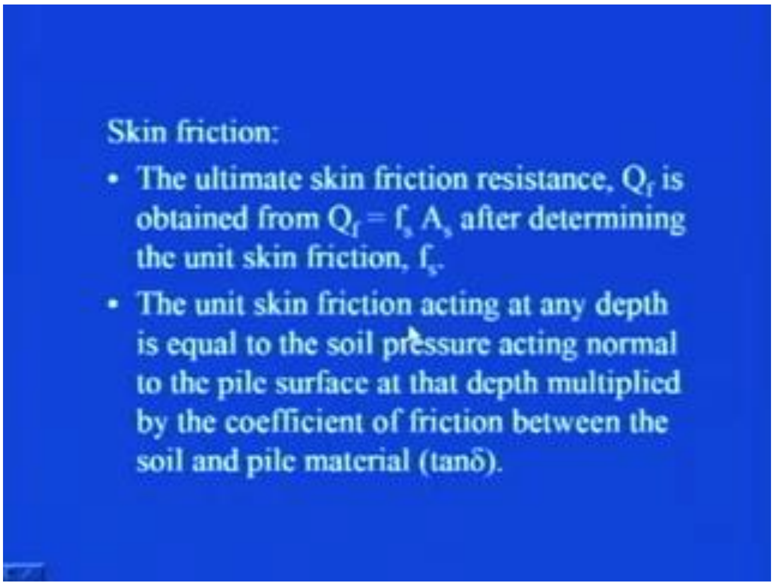

Now, how you can determine this unit skin friction f s, let us see that the unit skin friction acting at any depth is equal to the soil pressure acting, normal to the pile surface at that depth multiplied by the coefficient of friction between the soil and the pile material. Let us try to see this once again; the unit skin friction acting at any depth is equal to the soil pressure acting normal to the pile surface, so pile surface is vertical.

So, whatever is the pressure which is acting normal to that particular surface, if you simply multiply that particular pressure by this coefficient of friction between soil and the pile material. That is let us say tan of delta, if delta is the angle of friction between the soil and pile material, then this coefficient of friction will be equal to tan delta. So, you simply multiply this by the stress or the soil pressure, which is acting normal to the pile surface at that particular depth, you will be getting this unit skin friction.

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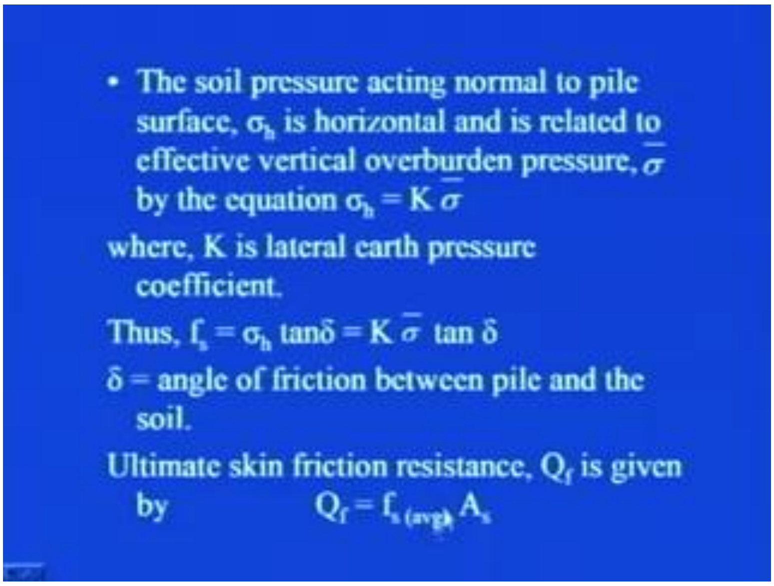

Let us try to see how we can do that in mathematical form, that the soil pressure acting normal to pile surface, say sigma h is horizontal, because the pile surface is vertical. So, it is normal will be horizontal and is related to effective vertical overburden pressure sigma bar, by the equation sigma h is equal to K into sigma bar. I hope that you remember, when we were discussing that lateral earth pressure theories in that we discussed that what exact, how you can find out the horizontal stress corresponding to the vertical stress at any particular depth z.

So, that is what is exactly here we are doing, where K is the lateral earth pressure coefficient, you simply multiply this K by effective over vertical overburden pressure and you will be getting this horizontal soil pressure acting at that particular depth. So, this f s will become this sigma h into tan delta, which will result into this particular expression; that is K into sigma bar tan of delta, where delta is the angle of friction between pile and the soil.

So, the ultimate skin friction resistance, this Q f is equal to f s average into A s, see why I am writing this f s average, when we were discussing that how the load from the super structure is getting transferred to the skin friction. That we saw, that earlier, it will just mobilize the skin friction to a particular length of the pile and as that applied load will go on increasing the length will go on increasing towards the tip of the pile.

So, at any particular point of time, this f s is not constant along the length of the pile and that is why, we are taking an average value, which I am representing as f s average into A s.

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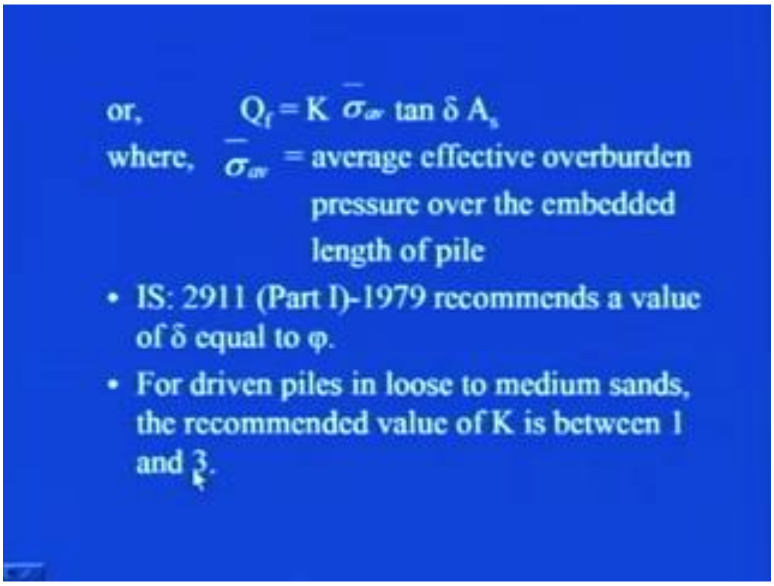

So, here this q f will become here we talked of this Q f which is f s average into A s, then f s average I can write in this using this particular expression, which will result into this Q f, which is equal to K sigma bar average tan delta into A s. Where, sigma bar average is average effective overburden pressure over the embedded length of pile, as I told you that at any particular point of time, this skin friction resistance is not constant throughout the length of the pile, so we have to take an average value of the same.

So, for that we take the average effective overburden pressure, because K is depending on the property of the soil, this K value will be constant at any particular depth, but this sigma bar average will be changing. So, we are taking this effective pressure as the average one over the embedded length of pile, then IS 2911 part I, 1979 recommends a value of delta is equal to phi.

So, whatever is the angle of internal friction of the soil, you can assume the friction angle between soil and pile material to be equal to the angle of internal friction of the soil. For driven piles in loose to medium sands the recommended value of K is in between 1 and 3, you must be thinking that, why the value of k is quite high in this case. You see, when we were discussing that active and passive earth pressure theories, we saw that in case of active condition, the coefficient of lateral earth pressure value was relatively much less as compared to the coefficient of lateral earth pressure in case of the passive condition.

You just imagine a situation, that a pile is getting driven into soil, what will be the case in that particular thing, whether it will be active case or whether it will be passive case. You see, you are driving the pile in the soil; the soil is having a tendency to move away from the soil. So, there the generation of passive condition is there and that is why such higher value of K is obtained.

So, the IS code is recommend IS code recommends the value of K between 1 and 3 in case of driven piles in loose to medium sands. However, the value of K and delta for piles driven into sand given by Broms 1966, they are as follows.

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Values of K and δ for piles driven into sand
(Broms 1966)

Pile material	δ	Values of K	
		Loose sand	Dense sand
Steel	20	0.5	1.0
Concrete	$0.75\ \varphi$	1.0	2.0
Timber	$0.67\ \varphi$	1.5	4.0

For pile material, that is steel delta in degrees that is 20 degree, for loose sand the value of K is 0.5, for dense sand it is 1, for concrete, it is three-fourth of the angle of internal friction of soil. And then, in case of loose sand the value of K is 1; however, in case of dense sand, this value of K is equal to 2. In case of timber piles, this value is 0.67 times phi, that is the angle of friction between soil and pile material is equal to 0.6 times the angle of internal friction on the soil.

And then, the value of K for loose sand and dense sand are one 0.5 and 4 respectively, this is as per Broms. However, IS code recommends that you can pick the value between 1 and 3 and you can take the value of this friction angle between pile and the soil to be equal to angle of internal friction of the soil, although it is clear from the expression, that for unit skin friction resistance f s, that it increases continuously with depth.

In reality, is it does so only up to the critical depth which varies from 15 to 20 times the pile diameter.

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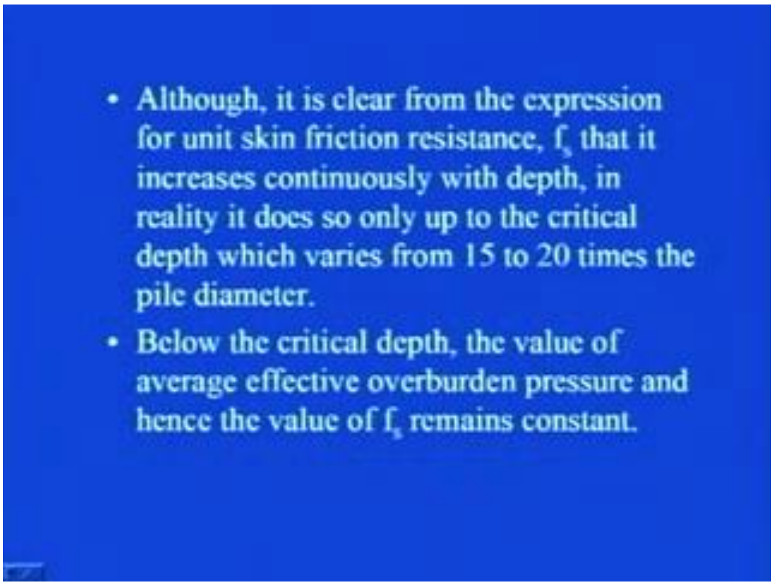

So, the critical depth concept, which I explained you earlier, is valid for this determination of unit skin friction resistance along with the unit point bearing resistance. So, exactly in the similar manner as we took here in case of point bearing resistance of this critical depth concept, we have to take care here in this case of unit skin friction resistance.

So, below the critical depth the value of average effective overburden pressure and hence the value of f s remain constant, because K is constant throughout the depth and tan delta will also be constant. So, is the only thing which was varying with depth was average effective overburden pressure and beyond critical depth, if and since this is constant, so this f s is also going to be constant.

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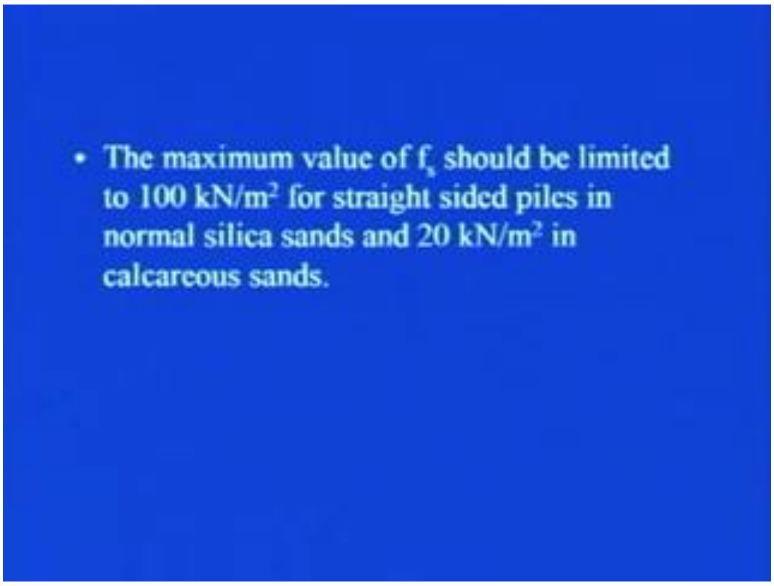

So, the maximum value of f s should be limited to 100 kilo Newton per meter square for straight sided piles, in normal silica sands and 20 kilo Newton per meter square in calcareous sands. So, these are just the recommendation; that we need to keep in mind, while estimating the unit skin friction resistance, if the unit skin friction resistance is getting increased to these values say 100 kilo Newton per meter square for normal silica sands.

Then, in that case you have to restrict that to 100 kilo Newton per meter square, this is the maximum value that you can take as far as the values of f s is concerned. Now, this was all about the driven piles, let us try to see some of the salient features of bored cast-in-situ piles and how we can estimate point bearing resistance and skin friction resistance, in case of bored cast-in-situ piles.

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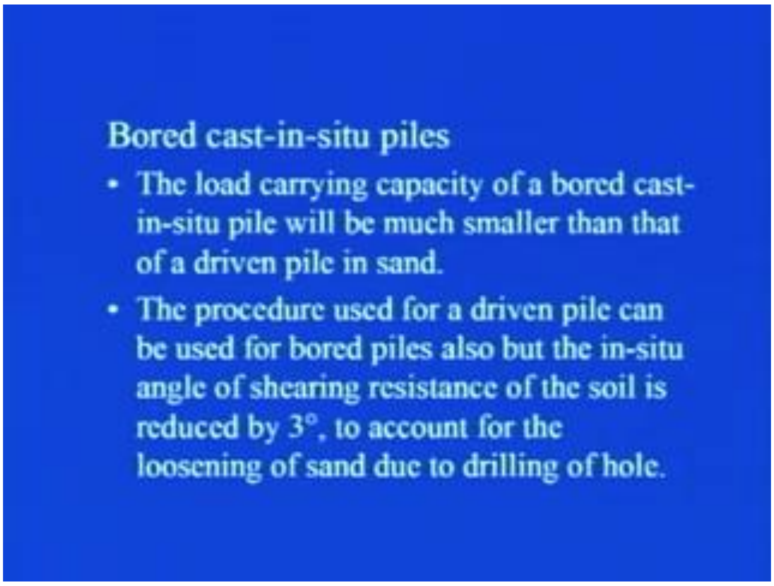

The load carrying capacity of bored cast-in-situ piles will be much smaller than that of driven piles in sand, due to the fact that because, in case of bored piles, you know that the void is created and then the concreting is done. So, in that case the active condition is getting generated and so the value of K becomes very less as compared to the value of K in case of passive condition which was relevant in case of driven piles in sand.

So, that is why this load carrying capacity of a bored cast-in-situ pile is much smaller as compared to driven piles in sand. So, the procedure used for a driven pile can be used for bored piles also, but the in-situ angle of shearing resistance of the soil is reduced by 3 degree to account for loosening of sand due to drilling of the hole. See, in case of driven piles, we modify the angle of in-situ friction angle as that was phi 1 plus 40 degree by 2.

However, in this case we simply have to reduce this phi 1 to by 3 degree and then whatever is the resulting angle of friction; correspondingly you can pick the values of bearing capacity factors. And then, go ahead for finding out the different terms as we discussed in case of driven piles.

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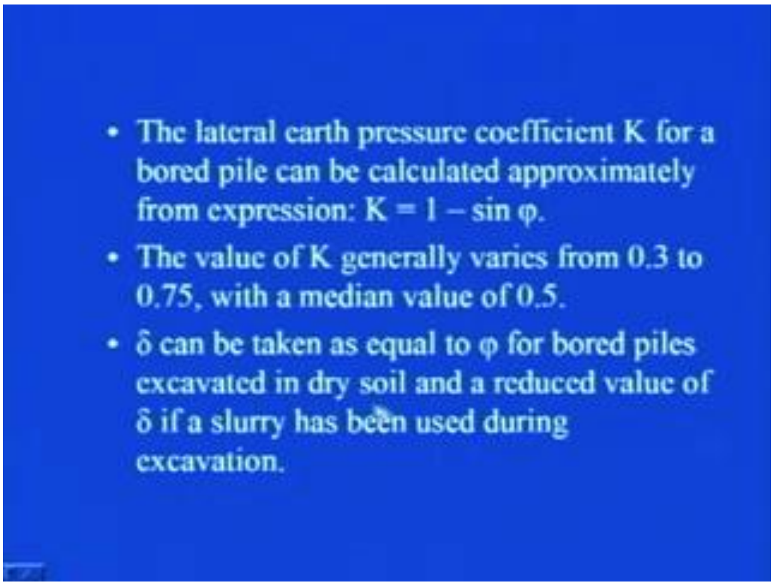

The lateral earth pressure coefficient for a bored pile can be calculated approximately from the expression; that is K is equal to 1 minus sin phi. So, if phi 1 is the angle of in-situ angle of friction, first you have to reduce that by 3 degree and then use that phi to go for any further calculation, so that same phi will be used over here. The value of K generally varies from 0.3 to 0.75 with a median value of 0.5. So, you see it was 1 to 3 in case of driven piles; however, it is 3 to 0.75 in case of bored piles.

Such a large difference and which results into the more load carrying capacity in case of driven piles, delta can be taken as equal to phi, for bored cast situ piles, driven in dry soil and at reduced value of delta, if a slurry has been used during the excavation. So, you see in bored in bored piles, you have to create some excavation and if you are using a slurry to stabilize or making the having facilitate the boring.

In that case, you can reduce the value of delta; otherwise you can take delta to be equal to phi for bored piles excavated in dry soils.

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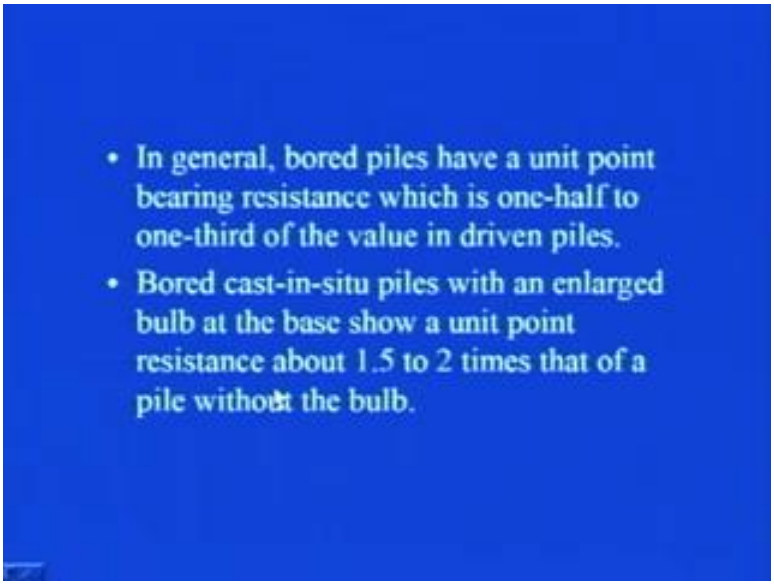

In general, bored piles have a unit point bearing resistance which is one-half to one-third of the value in driven piles. So, let us say that you have found out the capacity under compression of a pile in case of driven piles, so and if you work out for bored piles. So, just to provide a check that what should be the order of this. So, this just gives you a rough guideline.

Bored cast-in-situ piles with an enlarged bulb at the base show a unit point resistance about 1.5 to 2 times that of pile without the bulb. Obviously, when we discuss that under reamed pile, then we saw that how exactly this bulb enlarged bulb can be formed at the tip of the pile or at the base of the pile. So; obviously, if a bulb is there which will result into larger point, bearing resistance.

And if you compare the two cases, that a pile with bulb and without bulb, then this much difference in the unit point resistance, that you can expect in the two cases, that is about 1.5 to 2 times that of pile without the bulb. Now, let us try to see that driven cast-in-situ piles, first we saw that how the driven piles work or how we can find out; it is load capacity under compression. Then, we saw that bored cast-in-situ piles, now the third one is driven cast-in-situ piles.

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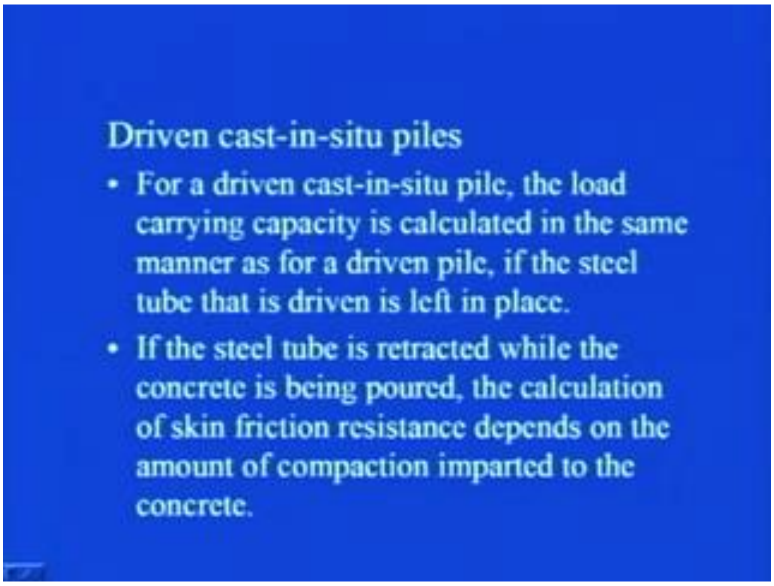

For a driven cast-in-situ piles, the load carrying capacity is calculated in the same manner as was for driven piles, if the steel tube that is driven is left in place, that is in case of the cased piles. If the steel tube is retracted, while the concreting is being poured or the concrete is being poured, the calculation of skin friction resistance depends on the amount of compaction imparted to the concrete.

So, you see there is a difference between cased and uncased piles, while you find out the load carrying capacity, in case of driven cast-in-situ piles. Let us, try to see that what are the other aspects related to this.

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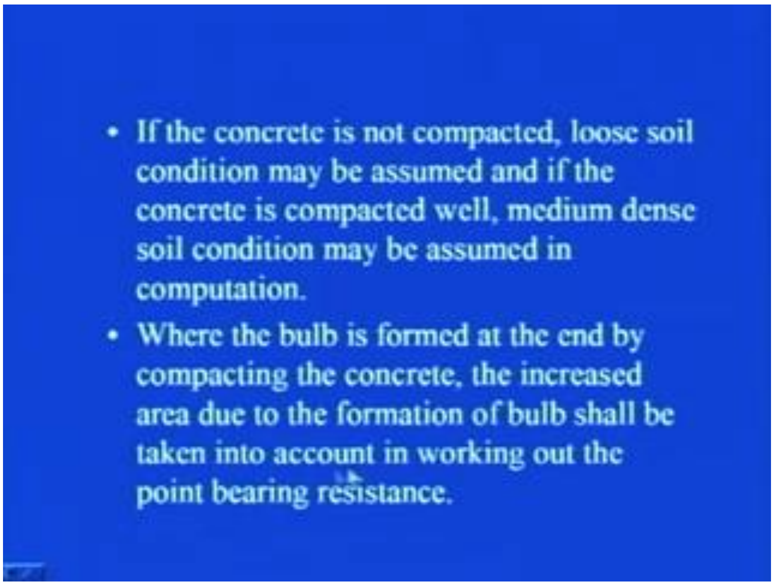

If the concrete is not compacted, loose soil condition may be assumed and if the concrete is compacted well medium to dense soil condition may be assumed in the computation. See, when you will be picking the value of say delta or N or K; there you will require that what exactly is the type of the soil or what is the condition of the soil. So, here in case the concrete is compacted, you have to assume loose soil is not compacted and then you have to assume loose soil condition.

And in case, the concrete is compacted well, you can use or assume medium dense soil condition and then can choose appropriate parameters. When the bulb is formed at the end by compacting the concrete, the increased area due to the formation of bulb shall be taken into account in working out the point bearing resistance. Obviously, that is why the bulb is being provided at the base of the pile.

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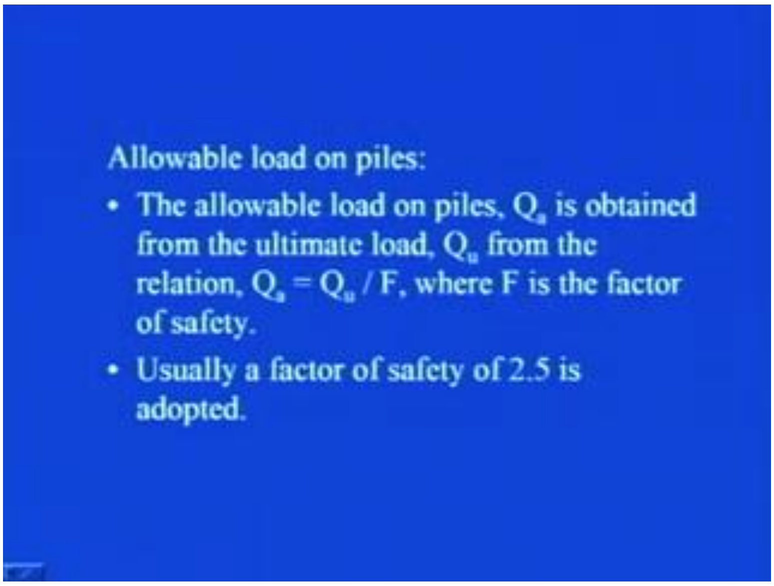

Now, what are the allowable load on piles, the allowable load on piles, that is Q a is obtained from the ultimate load Q u from the relation, which is Q a is equal to Q u by F, where F is the factor of safety. So, you see we can find out the ultimate point bearing resistance and the ultimate skin friction resistance. We can find out the load corresponding to these resistance; if we add them up we get the ultimate load capacity of the pile. Simply, divide that by factor of safety, you will be getting that how you can find out allowable loads on pile.

Usually, a factor of safety of 2.5 is adopted, then this was all about that we were discussing that when the piles they are driven or bored in the sand or granular material what about in clays. So, let us try to see some of the aspects related to piles in clay.

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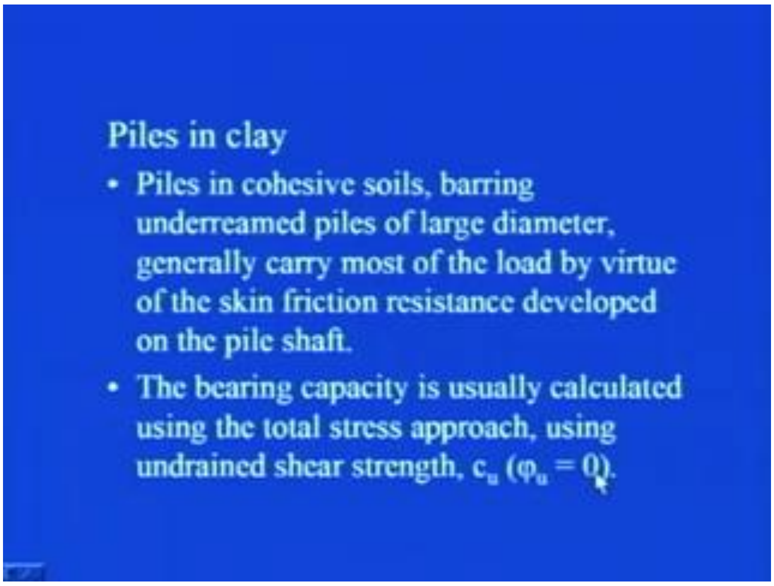

Piles in cohesive soils, barring under reamed piles of large diameter, generally carry most of the load by virtue of the skin friction resistance developed on the pile shaft. So, you leave apart, the option of under reamed pile, where the bulbs are provided to increase point bearing resistance. Apart from those, in case of piles in clays, usually whole of the load or most of the load which is coming from the super structure is getting resisted by the friction, which is mobilized along the pile shaft or by skin friction resistance.

The bearing capacity is usually calculated using the total stress approach using undrained shear strength and in this case since it is cohesive material, purely cohesive material. So, undrained angle of internal friction will be equal to 0, which I am showing here that phi u is equal to 0, so we have to use this total stress approach.

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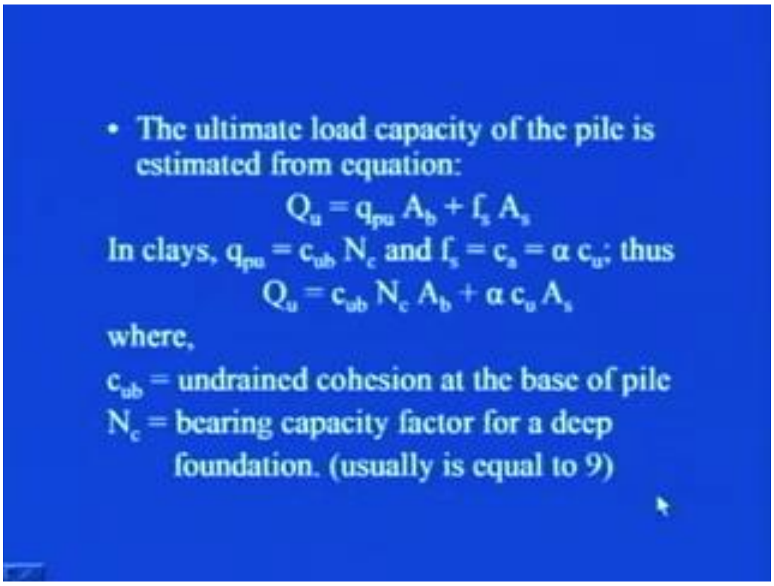

So, the ultimate load capacity of the pile is estimated from the equation, exactly in the similar manner this general equation is there, that is Q u is equal to q p u, which is acting over the area A b, which is area of the base of the pile plus f s into A s. In case of clays, this q p u is equal to c u b into N c, that we have already discussed and f s is equal to the adhesion between the pile and the soil, which is equal to c a, c subscript which is equal to alpha times c u, that is the undrained cohesion.

Thus, this particular equation will result into this Q u is equal to c u b into N c which I am writing from this particular expression into A b plus, this f s is equal to alpha times c u. So, this will result into alpha c u into A s, where c u b is undrained cohesion at the base of the pile, N c is bearing capacity factor for a deep foundation, usually it is taken to be equal to 9.

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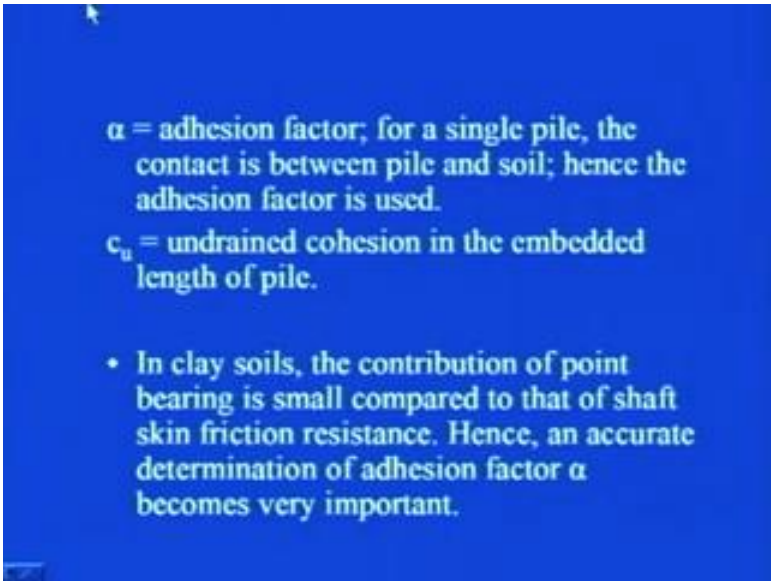

Alpha is adhesion factor, for a single pile the contact is between pile and soil and hence the adhesion factor is used, why not you must be thinking that, why not the coefficient of friction, but here that if a single pile is there which is a contact between the pile and the soil. So, this adhesion factor is being used, c u is undrained cohesion along the embedded length of the pile.

In clay soils, the contribution of point bearing is small compared to that of shaft skin friction resistance; hence an accurate determination of adhesion factor becomes very important. You see, this part is quite negligible in case of this pile in clay, because most of the load which is coming from the super structure to the pile is being mobilized through this particular term.

So, we really need to estimate this alpha properly, so that is why, it is written here that an accurate determination of this adhesion factor alpha is very important.

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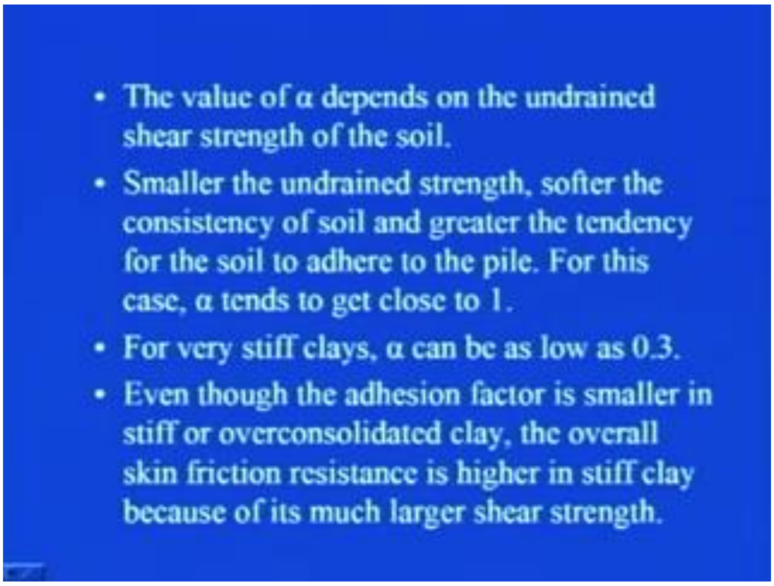

Now, how we can estimate these values of alpha, the values of alpha depends on the undrained shear strength of the soil, smaller the undrained strength, softer the consistency of soil and greater the tendency for the soil to adhere to the pile. For this case alpha, will tend close to 1, so you see for smaller undrained shear strength, what will happen, the soil will be more soft in nature and as the soil is more soft, since it is clay, as the clay is soft, it is consistency is low. Then, it will have the greater tendency to adhere to the pile shaft and so the alpha will tend to 1.

For very stiff clays, this alpha can be as low as 0.3, even though the adhesion factor is smaller in stiff or over consolidated clay, the overall skin friction resistance is higher in stiff clay, because of it is much larger shear strength. Because, you know that alpha times c u, so although the value of alpha is small, but being c u to be more in case of over consolidated or very dense clay, this skin friction resistance is higher.

So, you see Tomlinson in 1979 gave this particular guideline or this particular chart, such that to you can estimate this adhesion factor alpha. So, on x axis, you can see it is undrained strength c u, which is in kilo Newton per meter square; however, in on y axis, it is adhesion factor alpha.

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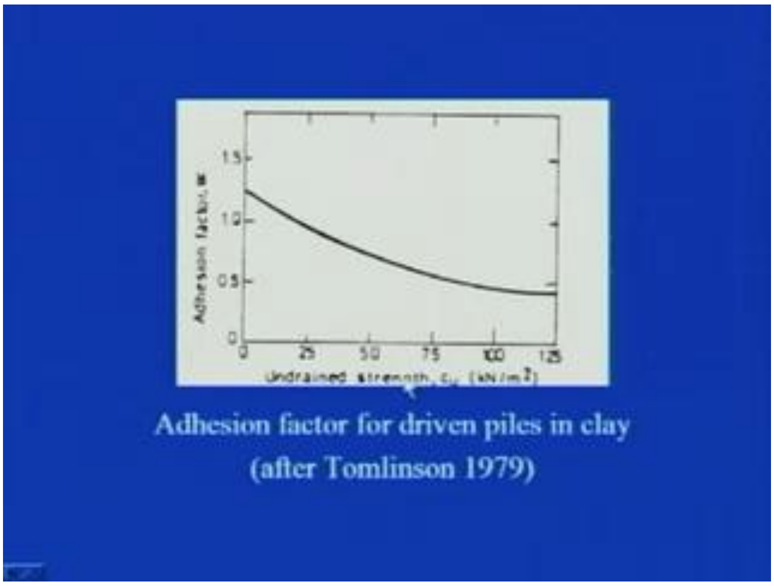

So, this is c u is the property of the soil, you know the property of the soil, so accordingly, you simply pick the value, let us say that the undrained strength of the soil is 25 kilo Newton per meter square. You simply pick this 25, draw a vertical line here, parallel to this y axis and simply you see here, it will cut the curve here at this particular point, it will intersect it here and simply you project that to this particular axis, that is on y axis.

So, correspondingly you can pick the value of this adhesion factor alpha corresponding to any particular value of undrained strength of the soil. Now, when the pile is driven into a soft soil stratum, the soil gets remolded and loses some of it is shearing strength.

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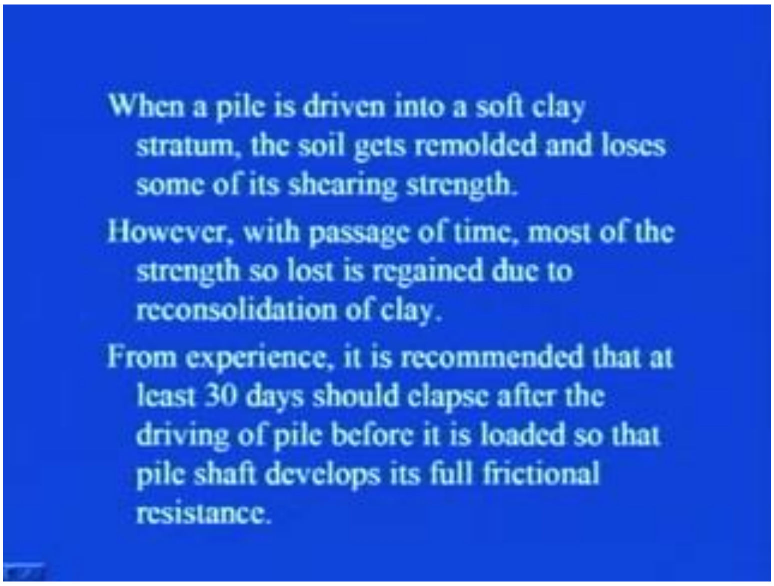

Since, we are talking of piles in clay, so we have to take care of these points, that what will happen when the pile is getting driven into the soft clay stratum, so in that case the soil will get disturbed and you have to take care of that in the analysis. However, with passage of time most of the strength, so lost is regained due to reconsolidation of clay, you know that in clay the consolidation is the predominant phenomena. So, with passage of time, this loss in the shear strength of the clay will be regained due to reconsolidation of clay.

So, from experience, it is recommended that at least 30 days should elapse, after the driving of pile, before it is loaded, so that pile shaft develops it is full frictional resistance. So, you have to give some time, so that the clay can regain it is strength, which it has lost during the process of driving, so pile is getting driven, the soil is getting loosened and remolded.

So, it is losing some of it is shearing resistance with passage of time it regains, so you have to give at least 30 days time, such that the soil can regain it is strength and it can develop, it is full frictional resistance.

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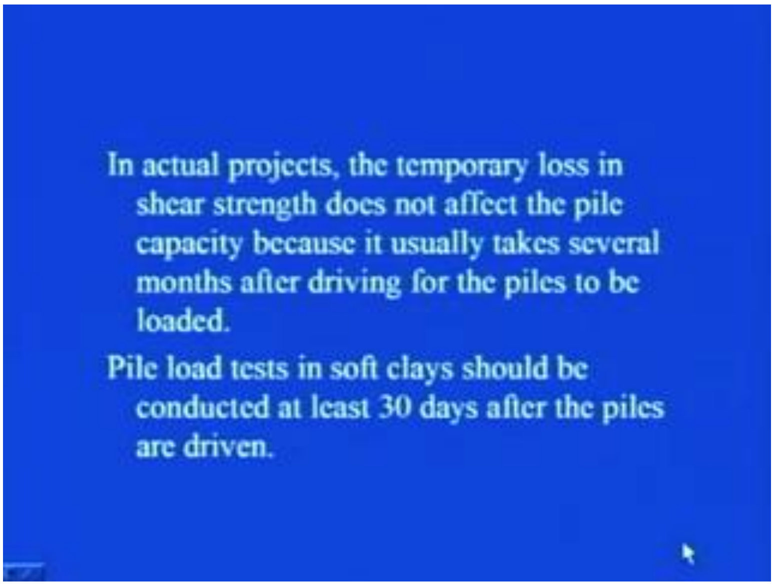

In actual projects, the temporary loss in shear strength does not affect the pile capacity because it usually takes several months, after driving for the piles to be loaded, as you have seen when we were discussing the construction of pile; it is a very long procedure. So, by the time super structure comes up or and the load is getting transferred to the piles, it takes several months. So, in during those months the soil regains it is strength, so in actual projects usually this temporary loss, we do not take into account.

Pile load test in soft clays should be conducted at least 30 days after the piles are driven, such that the piles can develop or fully mobilize the resistance, which it is supposed to offer, while working under the load which is coming from the super structure. Now, to determine skin friction capacity of bored and cast-in-situ piles, Tomlinson recommended an average value of alpha is equal to 0.45 in firm to stiff clays with an upper limiting value of 1 kg per centimeter square for unit skin friction resistance.

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So, these are some of the guidelines for picking the values of alpha, in fissured clays this value can be taken as 0.3. The recommended values of alpha for different consistencies of clay have been summarized in this particular table, which you can see here.

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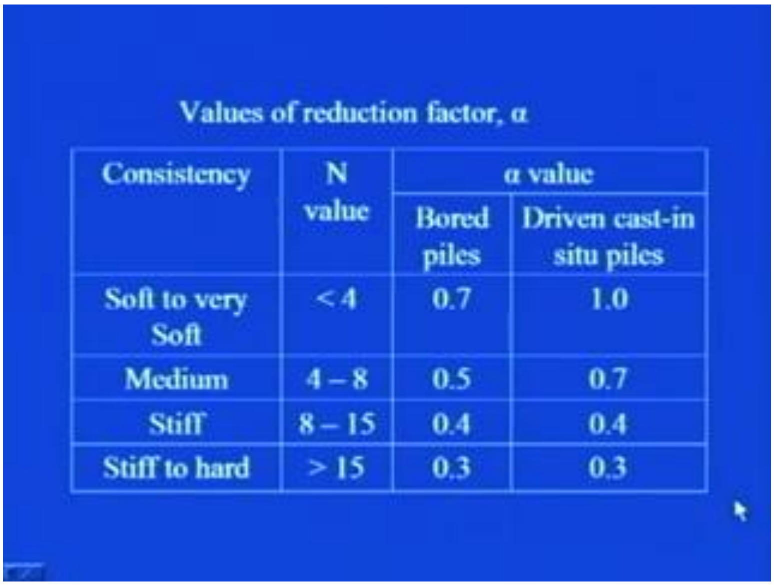

Values of reduction factor, α

Consistency	N value	α value	
		Bored piles	Driven cast-in situ piles
Soft to very Soft	< 4	0.7	1.0
Medium	4 – 8	0.5	0.7
Stiff	8 – 15	0.4	0.4
Stiff to hard	> 15	0.3	0.3

As I told you, that as the consistency of the soil is softer that is this, the value of alpha is greater and it is towards 1. So, you can see here that soft to very soft N values, they are less than four N is standard penetration number; that you get from standard penetration test, it is less than 4, in case of bored piles, it is 0.7 and in case of driven cast-in-situ piles it is 1. In case of medium consistency of soil, where the N value is in between 4 to 8 bored piles the alpha value is 0.5; however, for driven cast-in-situ piles, it is 0.7.

And then, you can see for stiff and stiff to hard consistency of the soil, that is 8 to 15 or more than 15, for bored piles, this value of alpha is 0.4 and 0.3, for stiff to hard clay respectively and is the same for driven cast-in-situ piles also. So, this just give you rough idea, so here you see, if in case you have standard penetration test data, so from that you can simply pick that what exactly is the consistency of the clay.

And accordingly, you can pick the value of alpha, for bored piles or for driven piles as the case may be. Now, code recommends, that for working out the safe load a minimum factor of safety of 2.5 should be used.

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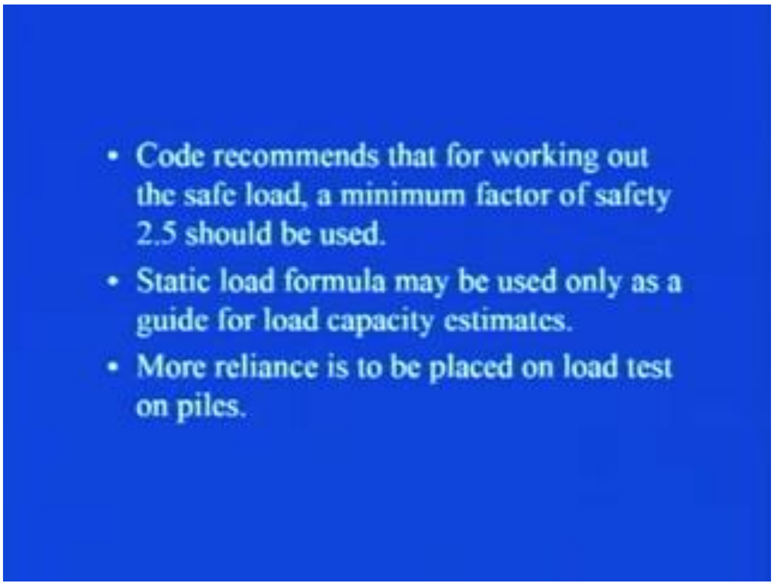

Then static load formula, may be used only as a guide for load capacity estimates, that is you really cannot rely on the on only the static load formula, more reliance to be placed on load test on piles. So, today we saw various aspects related to static pile load formulae; however, we have seen that, one should not solely rely on this particular type to estimate the load carrying capacity of the pile under compression.

So, we need to go for in-situ pile test and those we will be discussing in the next class and I will be sharing some of my experience at one particular site, where we conducted that pile load test. And, what are the various specifications of that particular various type of test; that has to be conducted in situ, that we will see in the subsequent lectures.

Thank you.