Soil Structure Interaction
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Lecture-59
Soil Structure Interaction for Pile Foundation

In my previous lecture, I discussed the procedure to determine the settlement of pile group in layered soil based on elastic analysis. Before I go to the third part, i.e., the determination of settlement of pile group based on the empirical expression, I want to mention about the correction factors that we are applying for elastic analysis.

For the interaction factor and R_s value, correction for Poisson's ratio should be applied if μ of the soil is less than 0.5. Along with this, correction should be applied if the pile has an enlarged base also. So these 2 corrections are recommended for floating piles. For the end bearing piles, these charts are not given. The Poisson's ratio correction factor values are very small compared to the other corrections factor values even if the spacing between the piles is very less, say S/d ratio of 5. So the correction factor for Poisson's ratio can be neglected for end bearing piles.

The correction factors for Poisson's ratio and enlarged base given for the floating piles can be used for the end bearing piles also. But if these corrections are being used for the end bearing pile groups, they should be applied for both group settlement calculation as well as the single pile settlement calculation. For example, when the problem on layered soil is being solved, corrections for Poisson's ratio and finite layer depth on R_s value should be applied. At the same time they should be applied on the single piles also.

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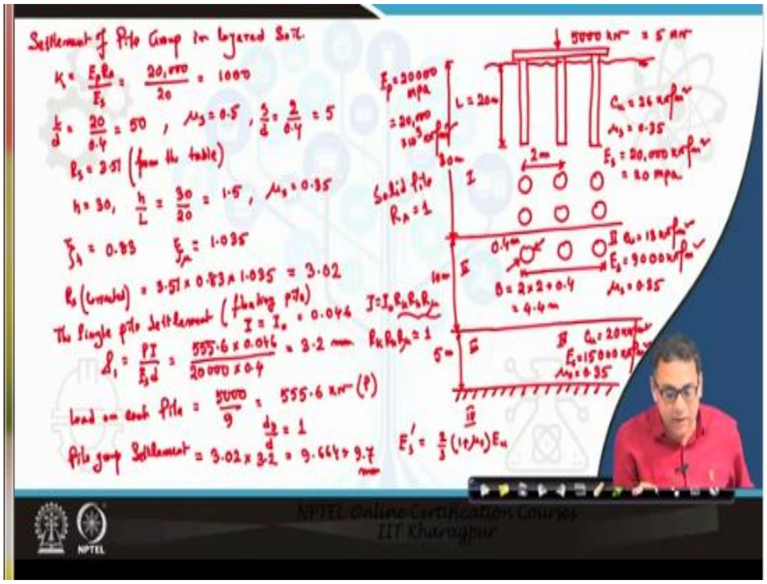

In the example solved in the previous class, it was mentioned that I = I_o = 0.046 which may imply that any corrections are not being applied. But in that particular case, the values of the correction factors are such that their product is almost equal to unity ($R_\mu R_h R_k \approx 1$). So, the corrections were applied in the previous case, but because of the correction factor values in that particular case, I and I_o turned out to be almost equal. So, it is very important to apply the correction factors for both I_o and R_s values.

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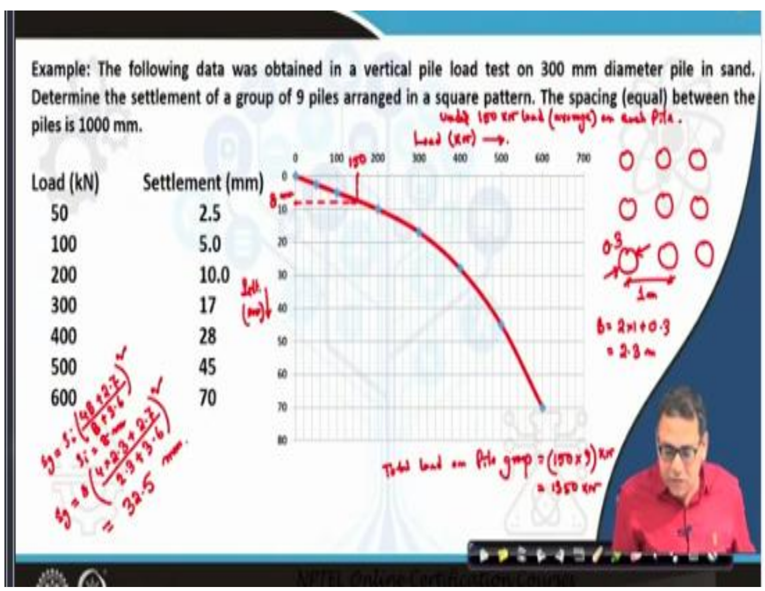

Now, let us see how to determine the settlement of the group pile using empirical expressions. The example problem shows a plate load test data for a 300 mm diameter pile in sand. The problem is to determine the settlement of a group of 9 piles arranged in a square pattern. The spacing between the piles is 1000 mm or 100 cm. Each pile has a diameter of 0.3 m and spacing between the piles is 1 m. So the width of the pile group considered as a block, B will be 2.3 m (2s+d = 2 × 1 + 0.3). If each pile carries a load of 150 kN, the settlement of the pile group should be determined. So the pile group carries a total load of (150 × 9) kN.

The plate load test data and chart given in the question are for the single pile. So consider the average single pile load of 150 kN. So the settlement of a single pile is 8 mm for a load of 150 kN. The expression given by Skempton to determine the group settlement is:

$$S_g = S_i\left(\frac{4B+2.7}{B+3.6}\right)^2$$

where, S_i is the single pile settlement.

$$S_g = 8\left(\frac{4\times 2.3+2.7}{2.3+3.6}\right)^2 = 32.5\,mm$$

This is the procedure to calculate the group settlement of piles using the empirical relation.

This is just another way to determine the settlement, but this is for sandy soil. If the soil is clay, the consolidation theory can be used to determine the settlement. Also, the procedure to calculate the settlement should be decided based on the parameters that are available. If parameters are available for more than one method, it is better to calculate the settlement through all the methods and the maximum value should be adopted out of them.

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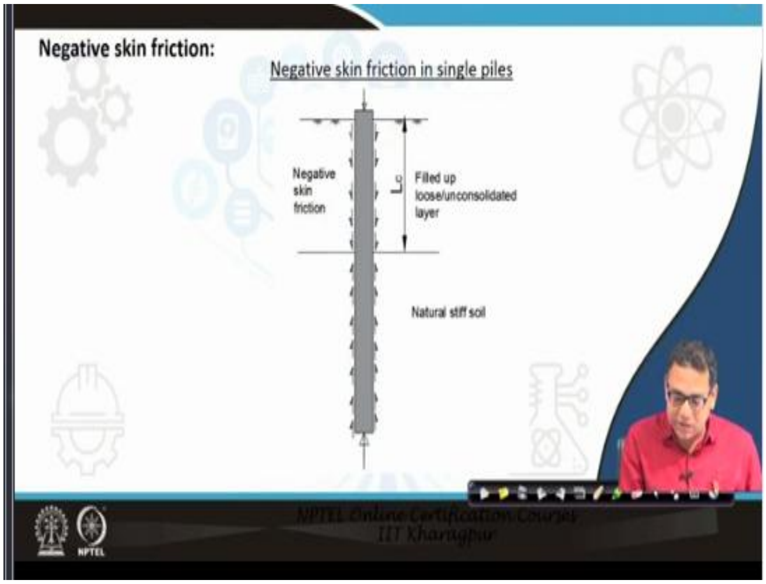

Now let us go through the concept of negative skin friction quickly. The negative skin friction is the case that occurs when the soil around a pile settles more than the pile itself. This occurs mostly when the surrounding soil is loose or is filled recently or is unconsolidated (if its) clay. If the surrounding soil is a loose sand or is recently filled, this phenomenon occurs because of the soil's own overburden pressure.

In the above example figure, the upper half of the surrounding soil shows that type of soil where the negative skin friction may occur. In such case, when the pile is subjected to a load, the pile displacement will be lesser than that of the soil.

Usually, the relative displacement of pile will be more meaning that the pile is going in the downward direction and the resistance from the surrounding soil will act in the upward direction. This is shown in the lower part of the figure above where a natural stiff soil is present. But if the surrounding soil is loose, then the entire mechanism described acts opposite. If the soil is loose, the relative displacement of the soil will be more which means that the soil moves in the downward direction compared to the piles. As the relative displacement of soil is more, it means that the pile is moving in the upward direction and so the resistance will act in the downward direction. Actually, both pile and soil are moving in the downward direction, but as the soil displacement is more compared to the pile, the resistance acts in the downward direction.

If the resistance acts in the downward direction, it is like the resistance itself is creating more imaginary load and hence the capacity of the pile reduces. This is the concept of negative skin friction. If this phenomenon is found anywhere, the negative friction or the negative resistance should be subtracted from the actual resistance that is offered by the stiffer part of soil.

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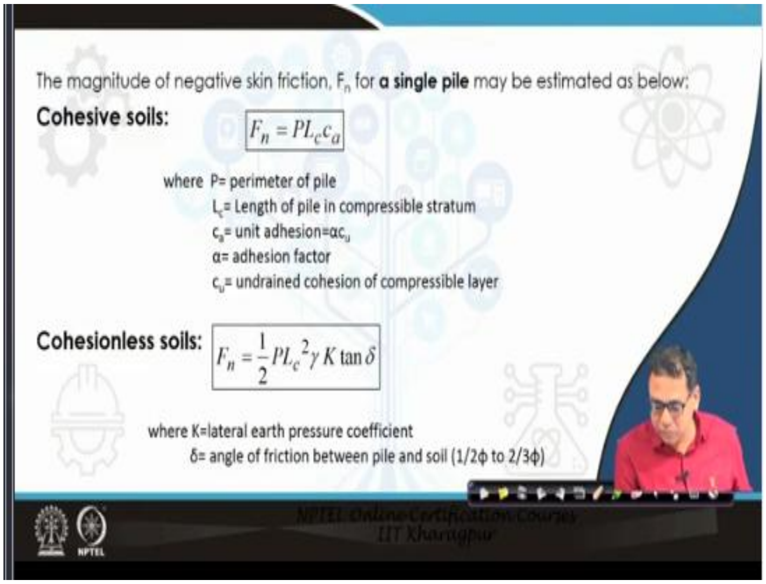

The calculation of the negative friction is same as that of the usual:

$$F_n = PL_c c_a$$

where, P is the perimeter of the pile, L is the length of the compressible zone, c_a is the adhesion factor which is equal to ($\alpha \times c_u$) where, α is the adhesion factor and c_u is the undrained cohesion.

Similarly for the cohesion less soils, the negative friction can be calculated by:

$$F_n = \frac{1}{2} P L_c^2 \gamma K \tan(\delta)$$

where, L_c is the length of the compressible strata, δ is generally taken as (1/2) φ to (2/3) φ and K is the lateral earth pressure coefficient.

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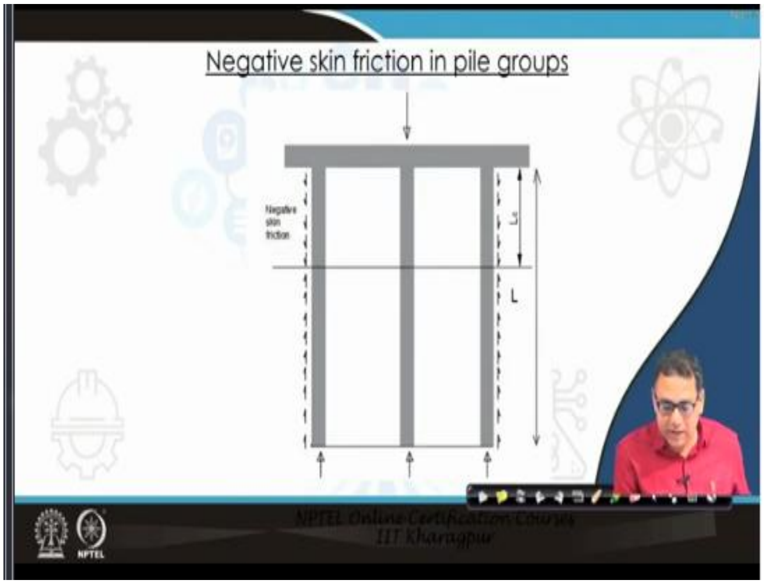

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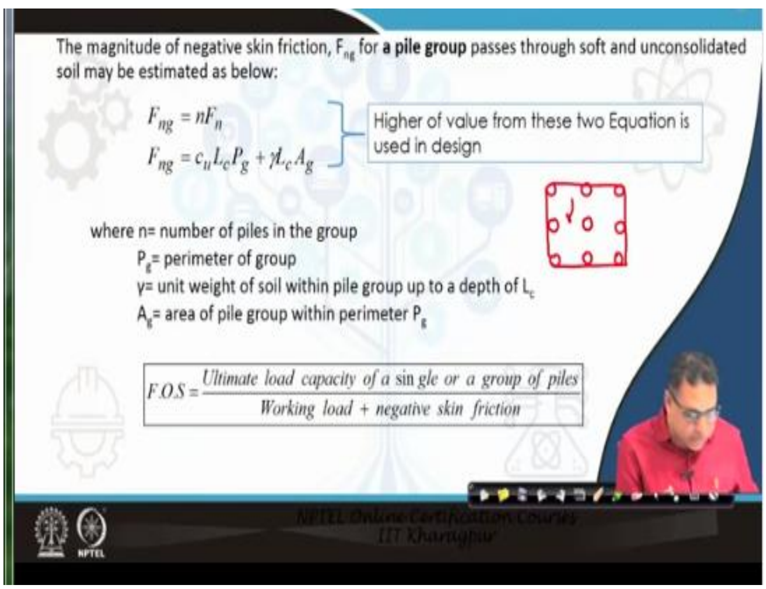

Similarly for the pile groups, the negative skin fiction can be calculated by calculating the single pile negative skin friction and then by multiplying it with the number of piles. After that, the negative skin friction should also be calculated based on the block failure which is same as the conventional pile block capacity calculation.

$$F_{ng} = nF_n \text{ (based on the single failure)}$$

$$F_{ng} = c_u L_c P_g + \gamma L_c A_g \text{ (based on the block failure)}$$

where, A_g is the area of the pile group within the perimeter P_g, L_c is the length of the compressible zone and γ is the unit weight of the soil within the compressible zone.

To determine the factor of safety when negative skin friction is in picture, the expression is given as:

$$\text{F.O.S} = \frac{\text{Ultimate load caryying capacity of single pile or pile group}}{\text{working load} + \text{negative skin friction}}$$

It is mentioned to subtract the negative skin friction from the resistance offered by the surrounding soil, but it is added to the working load in the above expression. Conceptually, both are the same because the applied load is being increased. So in an effective way some amount of resistance is being decreased due to the negative skin friction. So, either the resistance should be decreased or the load should be increased to incorporate the effect of the negative skin friction.

Then it should be compared the resistance offered by the natural soil and then the factor of safety should be determined. For that, the tip resistance should also be calculated which is necessary to calculate the ultimate load carrying capacity of the pile or the pile group. This is the end of the pile under compression or compressive loading. The next topic is the pile under lateral loading.

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In the lateral loading part, the settlement response and the ultimate lateral load carrying capacity will be discussed. Here, the pile settlement response is more important compared to the ultimate load carrying capacity or the ultimate lateral load capacity of the pile. So even though the discussion will be about both the aspects, the main focus will be on the settlement response of lateral loaded pile.

Also, the procedure to generate the p-y curve will be discussed (p is the stress and y is the displacement) for laterally loaded pile. Two approaches will be discussed in this regard. One is the subgrade modulus approach which was discussed in the shallow foundation also. The second approach is the elastic analysis that was discussed in the pile under comprehensive load.

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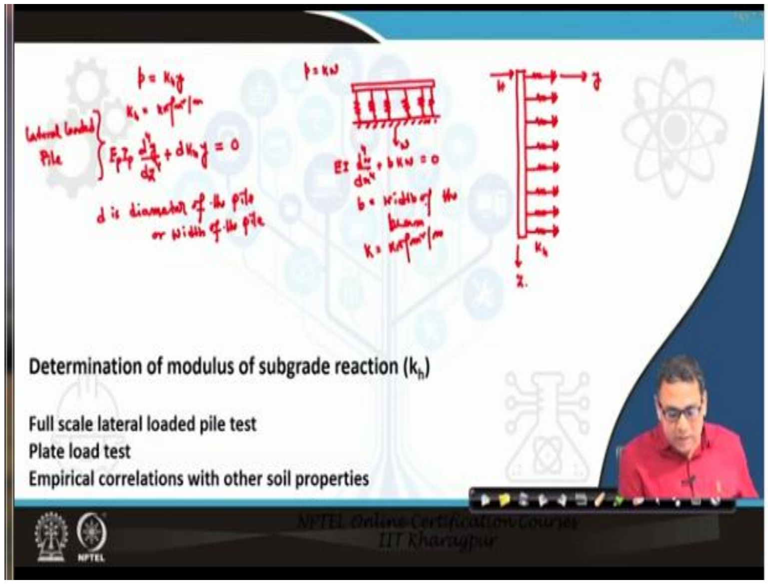

First let us see the subgrade modulus approach. Here the pile is analyzed as a beam. This is just like the shallow foundation case. So the soil, here, is represented by lateral springs but in the shallow foundation case vertical springs were used to idealize the soil. Consider that a lateral load of H is applied on the pile. For the linear analysis in the lateral direction, these spring constants are assumed to be k_h.

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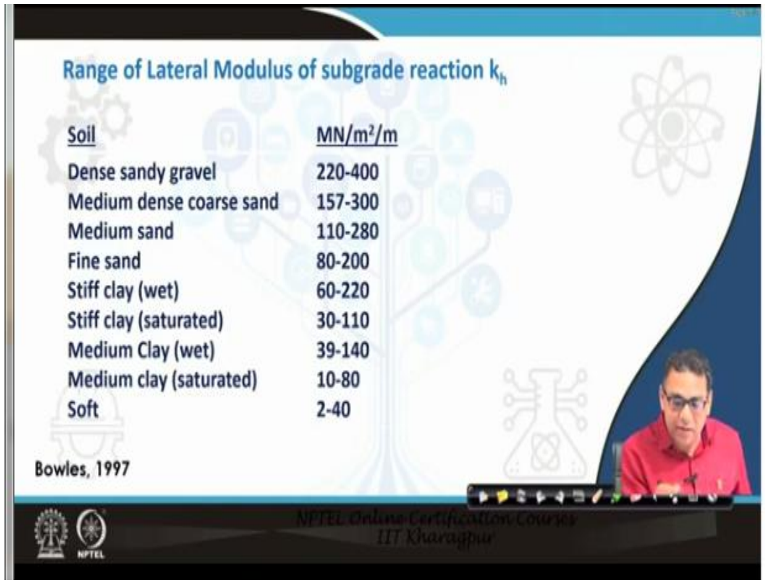

So, k_h is nothing but the subgrade modulus in the lateral direction or lateral modulus of subgrade reaction. So, the stress in the lateral direction will be equal to $k_h \times y$. Here, y is in the lateral direction and p is the stress acting on the pile. Here the k_h is the lateral subgrade modulus or the lateral modulus of subgrade reaction.

The expression of beam without any loading will be:

$$EI\frac{d^4w}{dx^4} + bkw = 0$$

where, b is the width of the beam and the units of k are kN/m^2/m.

Now, for the lateral loaded pile case, the expression would be:

$$E_pI_p\frac{d^4y}{dz^4} + dk_hy = 0$$

where, E_p is the elastic modulus of the pile, I_p is the moment of inertia of the pile, d is the diameter or width of the pile and the z direction is vertical. (units of k_h are kN/m^2/m)

In the initial lectures, the procedure to determine k value was discussed. Now the determination of k_h will be discussed. The possibilities are: a full scale lateral loaded pile test, plate load test and empirical correlations with other soil properties. For a full scale pile load test the pile should be instrumented. Based on that instrumentation the displacement corresponding to a particular stress can be calculated. So if the stress and displacement are measured, then this kind of relation: $p = k_h \times y$ can be developed from which k_h can be calculated.

This process is more time consuming, costly and we need sophisticated instrumentation to get that. So, sometimes the plate load test data or the empirical correlations of k_h with other soil properties are preferred.

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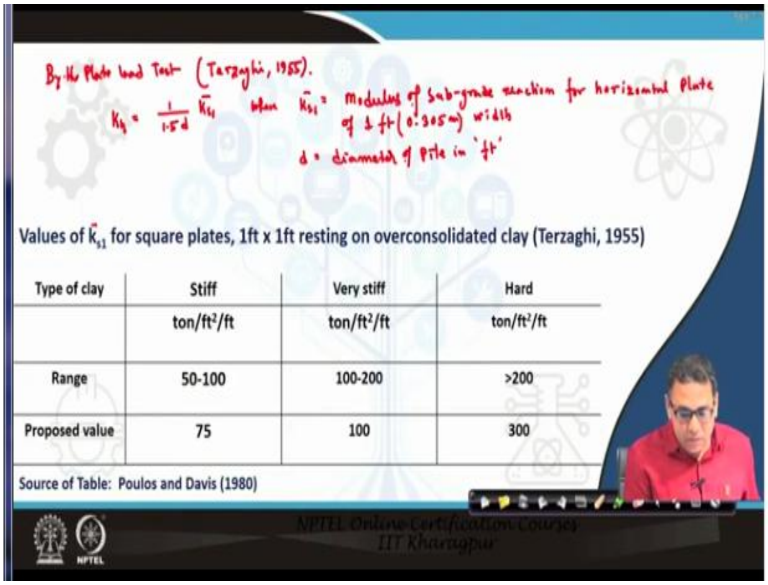

Speaking of empirical correlations, Terzaghi conducted a plate load test on a square plate of 1 ft × 1 ft and based on that, for different types of clay in over consolidated state, the values of k_h were suggested. The relation suggested by Terzaghi was:

$$k_h = \frac{1}{1.5d}\bar{k}_{s1}$$

where, $\bar{k}_{s1}$ is the modulus of subgrade reaction for horizontal plate of 1 ft (0.305 m) width and d is the diameter of the pile (in ft).

So, if the diameter of the pile is known, the k_h can be calculated for the soil.

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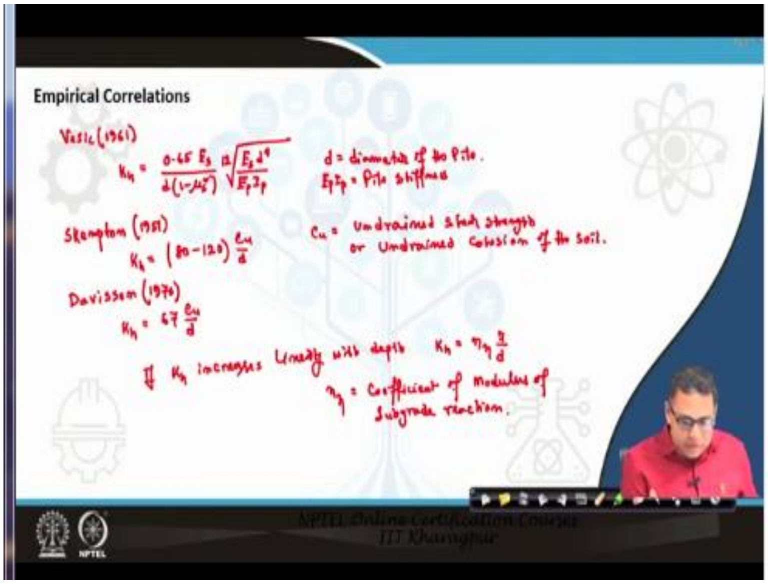

Now let us see a few empirical correlations quickly. Vesic in 1961 proposed the relation:

$$k_h = \frac{0.65E_s}{d\left(1-\mu_s^2\right)}\sqrt[12]{\frac{E_s d^4}{E_p I_p}}$$

where, E_pI_p is the pile stiffness, d is the diameter of the pile, μ_s is the Poisson's ratio of the soil and E_s is the elastic modulus of the soil.

The next correlation was proposed by Skempton in 1951:

$$k_h = \left(80-120\right)\frac{c_u}{2}$$

where, c_u is the undrained shear strength or undrained cohesion of the soil.

The next correlation was proposed by Davisson in 1970:

$$k_h = 67\frac{c_u}{d}$$

c_u is the undrained cohesion and d is the diameter of the pile.

Remember that for a stiff clay the k_h value will be more or less uniform. But for a sandy soil and for the soft clay, the k_h varies linearly with depth. So in general, for clay, the k_h value is considered to be uniform and for sandy soil k_h value is considered to be varying with depth or increasing linearly with depth. So, if the k_h varies linearly with depth then the expression will be:

$$k_h = \eta_h \frac{z}{d}$$

where, η_h is a coefficient of modulus of subgrade reaction. So, if k_h increases linearly with depth then k_h will be equal to z/d because it varies linearly (i.e., η_h = 1). If this is true for a soil, then the E value also increases linearly with depth mainly for the sandy soil. Later on the nonlinear pattern will also be showed.

As the overburden pressure and density increase with depth in most of the cases, the modulus value also increases. This modulus value is a function of stress especially in case of sandy soil. In clayey soil generally it is assumed to be uniform, but for the soft clay it varies.

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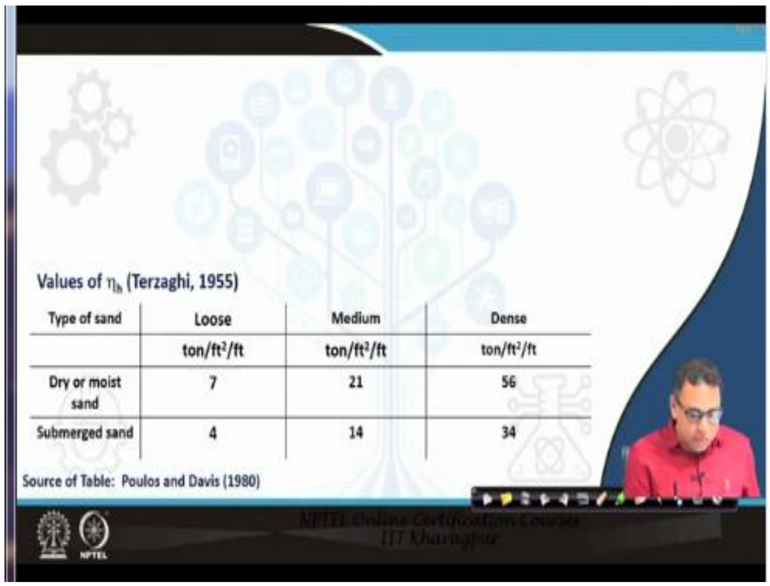

The above values are suggested by Terzaghi in 1955 for different types of sandy soils.

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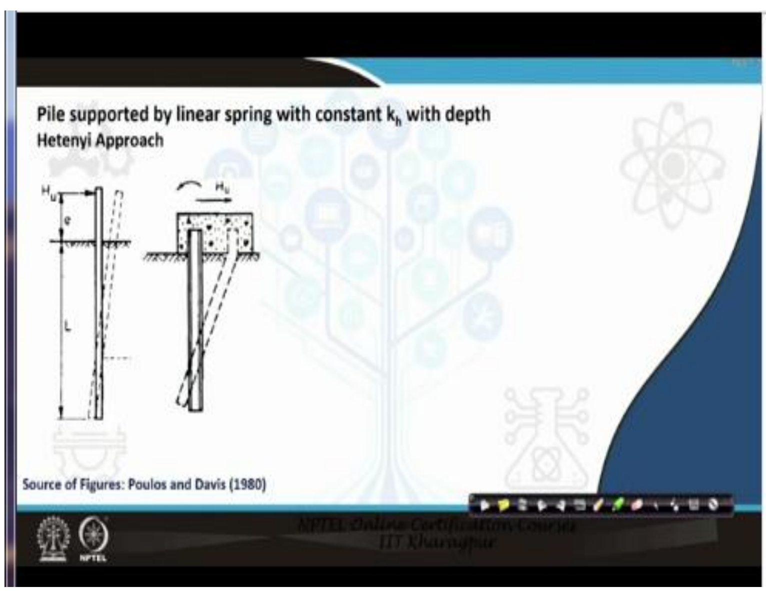

In the next class I will discuss how to determine the settlement of pile under lateral loading. After that I will discuss the subgrade modulus concept under two cases (constant k_h and varying k_h with depth) and then the elastic analysis. Thank you.