Rock Mechanics and Tunnelling
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Lecture 43
Foundations (contd.)

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Hello, everyone. I welcome all of you to the 2nd lecture of Module 9. So, in Module 9, we have started discussing about the foundations. And also, we will also discuss about the rock support systems in this module. In our previous class, we have started discussing about foundations. Today we will also discuss about the foundations only.

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So, today, we will begin with the estimation of safe bearing pressure from plate load test as per IS code guideline. Then we will discuss about different modes of failures of foundations in rock mass. Then estimation of ultimate bearing capacity of foundation on intact rock, then foundation on heavily jointed rock.

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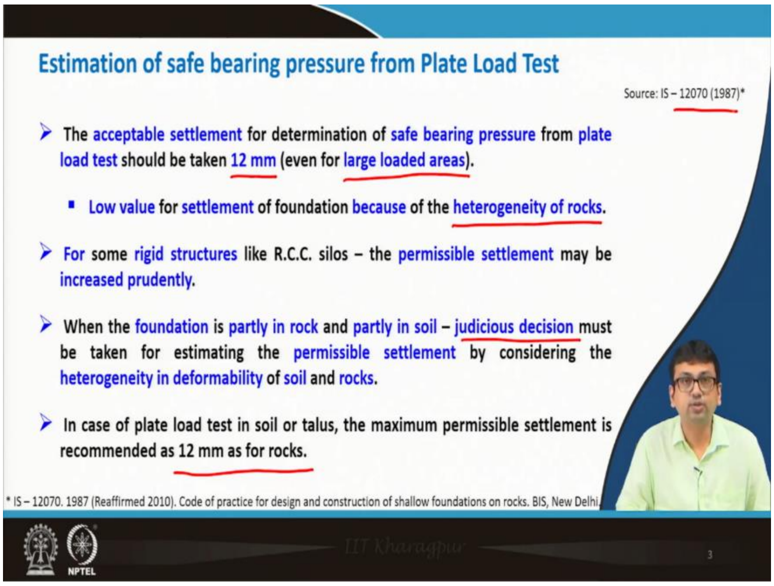

So, we will begin with the estimation of safe bearing pressure from plate load test. As per IS code guideline, the acceptable settlement for determination of safe bearing pressure from plate load test should be taken as 12 mm even for large loaded area. Now, the low value of settlement of foundation is generally considered because of the heterogeneity of rocks.

As we know that rock is highly heterogeneous and highly uncertain. That is why, it is better not to consider too much settlement even for large eluded areas. Because, maybe within very near vicinity, you may find a highly fractured weak rock. That is why, the low value of settlement of foundation should be considered because of the heterogeneity of rock mass.

For some rigid structures like R.C.C. silos, the permissible settlement may be increased judiciously which is up to the decision of the designer. Other than that, when the foundation is partly in rock and partly in soil, a judicious decision must be taken for estimating the permissible settlement by considering the heterogeneity in deformability of soil and rocks. So, what I mean to say that a part of the structure or foundation is on the rock and a part of the structure or foundation is on the soil and the rock and the soil are having different strength.

So, as a result of that judicious decision need to be taken for estimating the permissible settlement. So, that is what here it is stated. So, judicious decision needs to be taken by the designer. In the case of plate load test in soil or talus, the maximum permissible settlement is recommended as 12 mm as for rocks, i.e., it is recommended as 12 mm for soil also. So, all these things are as per IS code 12070 (1987).

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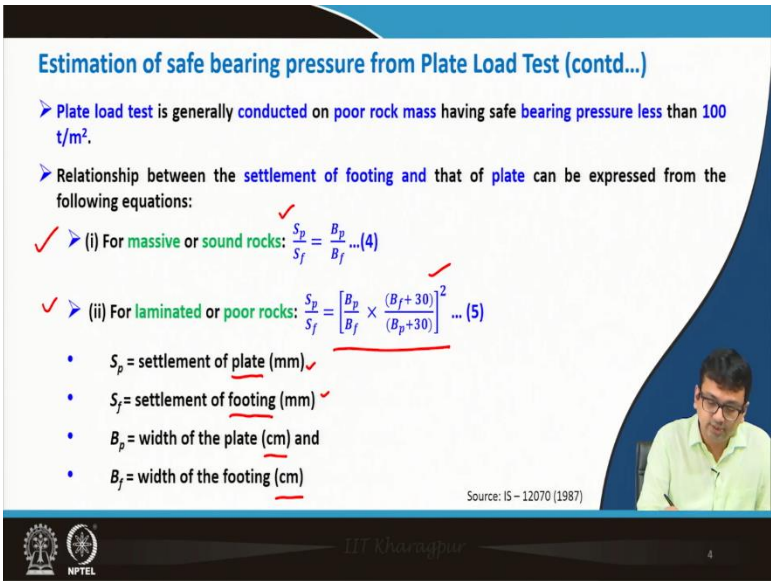

Now, the plate load test is generally conducted on poor rock mass as we have discussed in our previous lecture. So, the estimation of safe bearing pressure from plate load test is for poor rock mass. So, that is why, it is written over here that the plate load test is generally conducted on poor rock mass having safe bearing pressure less than 100 t/m^2. The relationship between the settlement of footing and that plate can be expressed from the following Eq. (4), for massive of sound rocks.

$\frac{S_p}{S_f} = \frac{B_p}{B_f}$. For laminated or poor rocks, $\frac{S_p}{S_f} = \left[\frac{B_p}{B_f} \times \left(\frac{B_f + 30}{B_p + 30}\right)\right]^2$ … (5). So, now, what are these terms? S_p is the settlement of plate, as you can see ‘p’ is the subscript. So, the settlement of the plate is in mm. S_f is the settlement of footing. That is why, the subscript is f. So, the settlement of the footing it is in mm. Now, B_p is the width of plate in cm.

The two settlement terms are in mm, the width terms are in cm, So, B_p, ‘p’ is the subscript and another term is B_f, width of the footing which is also in cm. So, the subscript is ‘f’. So, the width of the footing is denoted as B_f. So, all these terms are now known to us, but one thing we need to remember that the S_p and S_f are in mm, and B_p and B_f are in cm and then if we take convert the units in these mm and cm in this way, then only these equations will work and remember Eq. (4) is for massive or sound rock and Eq. (5) is for laminated or poor rocks.

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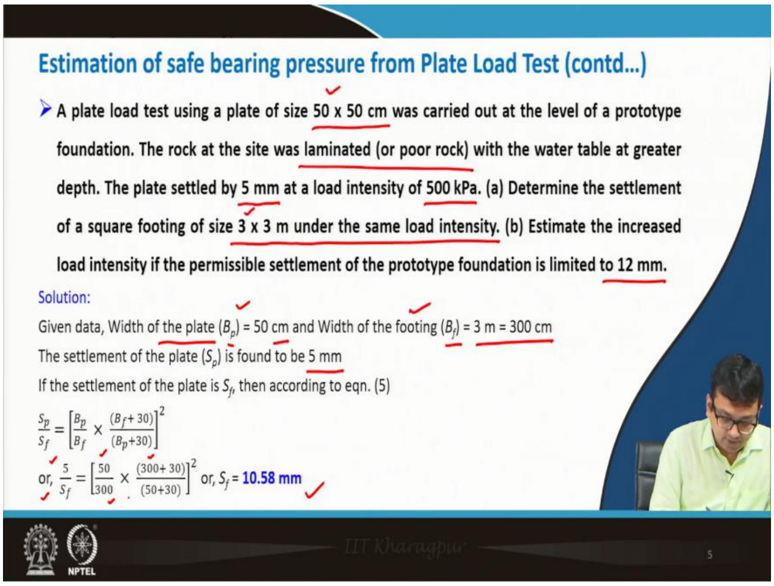

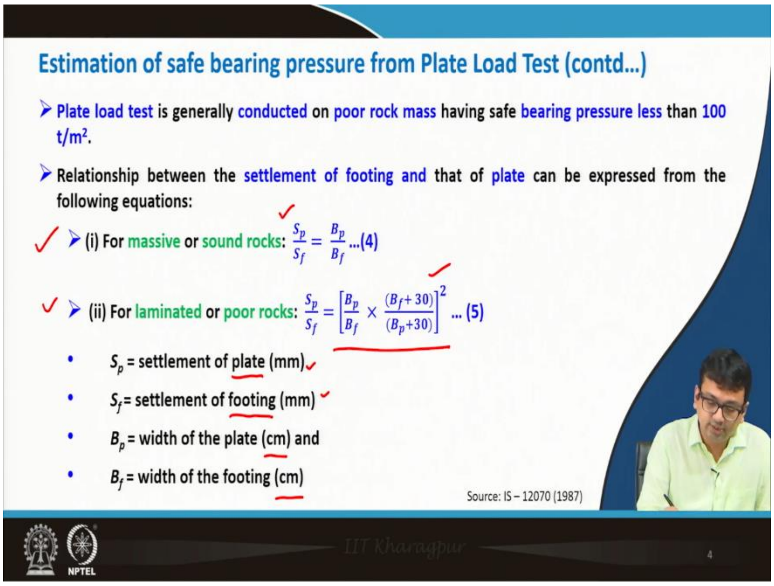

Now, let us solve again one problem and let us clear our doubts how to obtain the permissible settlement and also the load carrying capacity related to that. So, the problem statement is that a plate load test using a plate of size 50×50 cm was carried out at the level of prototype foundation. The rock at the site was laminated or poor rock with the water table at greater depth.

So, the rock is laminated or poor. The plate settled by 5 mm at a load intensity of 500 kPa. Now, determine the settlement of a square footing of size 3×3 m under the same loading intensity and then estimate the increased load intensity if the permissible settlement of the prototype foundation is limited to 12 mm. The rock is laminated or poor. So, which equation we will use? We will use the Eq. (5). Now, here you see what are the things given to us? Size of the plate is given, size of the footing is given and then the S_p is also given to us.

So, width of the plate (B_p) which is 50 cm and the width the footing (B_f) is 3 m that is given. So, we have to convert the unit of the width of the footing to cm as in the Eq. (5), the B_p and B_f are in cm and S_p and S_f are in mm. So, B_p is 50 cm and B_f is 3 m which is 300 cm. Now, the settlement of plate (S_p) is given as 5 mm.

So, now, we can get the S_f very easily. So, if the settlement of the plate is Sf, then using the Eq. (5), we can get this magnitude of S_f. So, $\frac{5}{S_f} = \left[\frac{50}{300} \times \left(\frac{300+30}{50+30}\right)\right]^2$.

So, you can get, S_f = 10.58 mm. So, this is the equation and we have used those input values and obtained the magnitude of S_f which is the settlement of the square footing of 3×3 m under the same loading intensity that is 10.58 mm. Next question is to estimate the increased load intensity, if the permissible settlement of the prototype foundation is limited to 12 mm. At this moment, the S_f is 10.58 mm.

Now, it is stating that estimate the increase load intensity if the permissible settlement of the prototype foundation is limited to 12 mms. But, at present, the settlement of the prototype foundation is 10.58 mm. So, we can allow a little more settlement also. Now, if we allow more settlement then obviously, the load intensity will increase.

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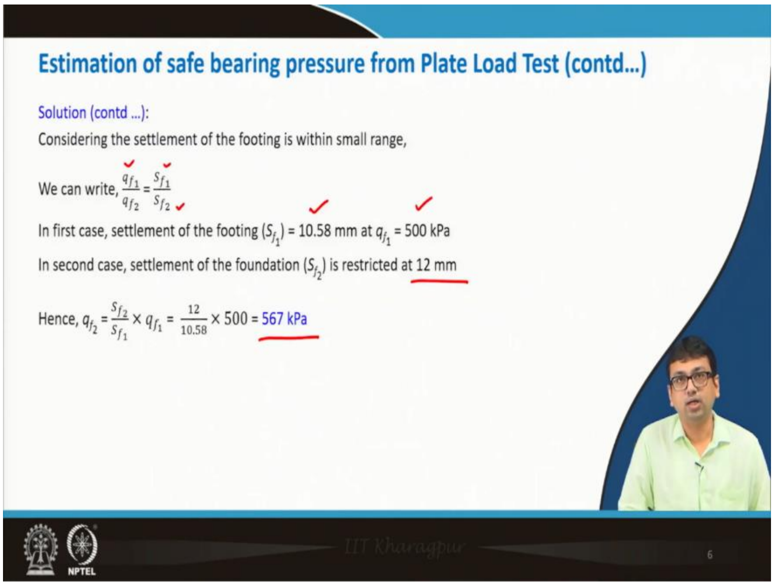

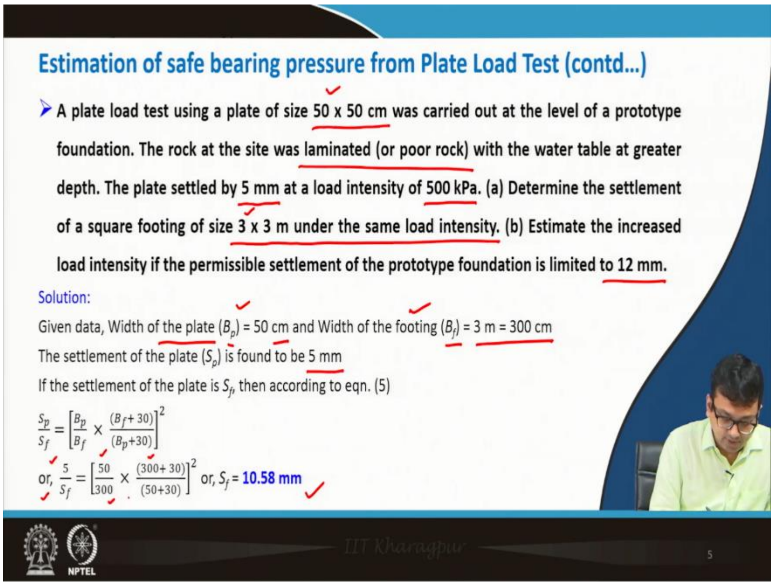

Thus, considering the settlement of the footing is within small range, we can say $\frac{q_{f1}}{q_{f2}} = \frac{S_{f1}}{S_{f2}}$.

Now in first case, the settlement of footing is S_{f1} what we have obtained as 10.58 mm and corresponding q_{f1} was 500 kPa. Now, for the second case, the settlement of foundation (S_{f2}) is restricted at 12 mm.

So, if I know S_{f1}, if I know S_{f2} and if I know q_{f1}, then we can get q_{f2}. So, q_{f2} = 567 kPa. So, this is the second answer. So, what was the second question? Estimate the increased load intensity if the permissible settlement of the prototype foundation is limited to 12 mm. So, what we are saying that the 10.58 mm settlement was for load intensity, 500 kPa. Now, the permissible settlement is allowed up to 12 mms. So, obviously, the load intensity will also increase so, now, it is becoming 567 kPa.

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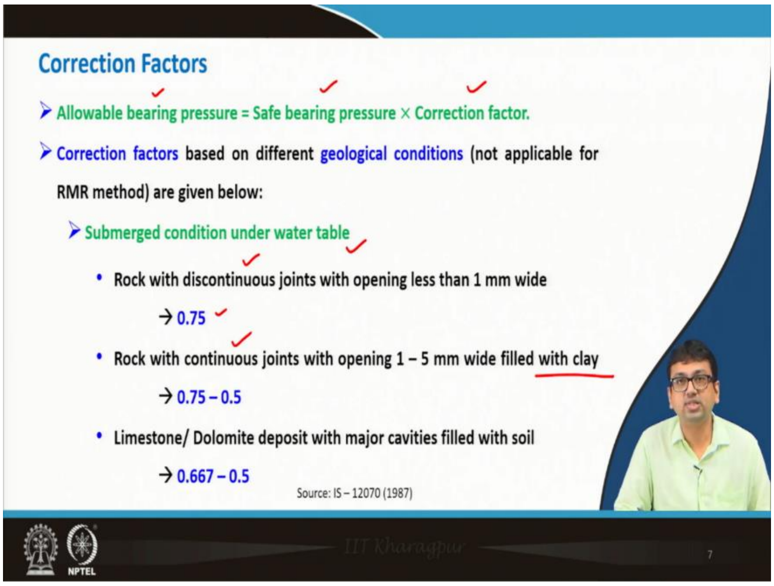

Now, again as per IS code guidelines, the correction factors are needed to find out the allowable bearing pressure. So, till now, we have discussed about the safe bearing pressure determination.

Now, in order to get the allowable bearing pressure, we need to multiply the safe bearing pressure with the correction factors. Now, the correction factors based on different geological conditions (which is not applicable for the RMR method). We have seen one table where we are getting the safe bearing pressure from the RMR. So, the correction factor based on different geological conditions (except RMR) are given below.

For the submerged condition under water table. Under that condition now, if the rock with discontinuous joints with opening less than 1 mm wide, the correction factor should be 0.75. Now, rock with continuous joints with opening 1 to 5 mm wide, filled with clay. So, then the maximum correction factor will be 0.75 and the minimum factor will be 0.5. Now, for limestone or dolomite, deposit with major cavities filled with soil, the correction factor will be 0.5 to 0.667.

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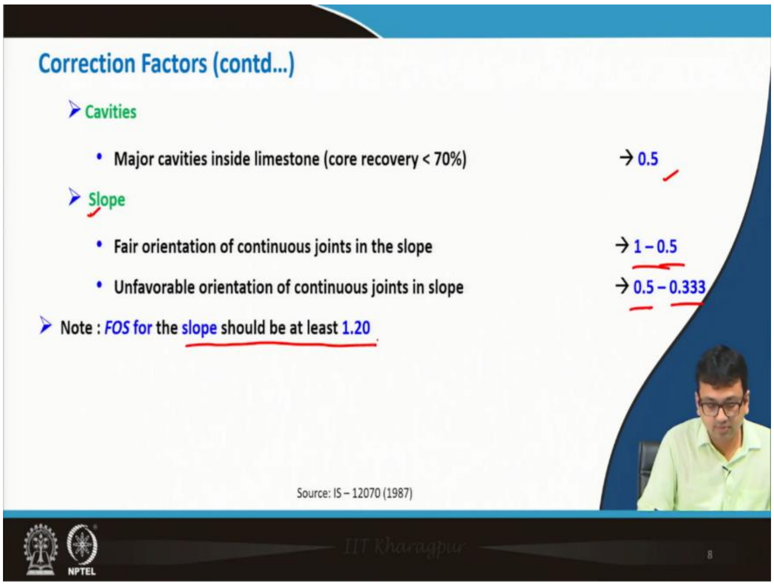

All these things are as per the IS–12070. Now under cavities, for major cavities inside limestone core recovery < 70%, the correction factor is 0.5. So, you have to multiply 0.5 with the safe bearing pressure to get the allowable bearing pressure. Now if the foundation is on slope, for the fair orientation of continuous joins in slope, the correction factor is from 0.5 to 1. If it is unfavourable orientation of continuous joints in slope, then the correction factor is 0.333 - 0.5. Now, it should be noted that the factor of safety for the slope should be at least 1.2. So, when we are discussing about slope, then IS code recommends that the factor of safety of a slope should be at least 1.2.

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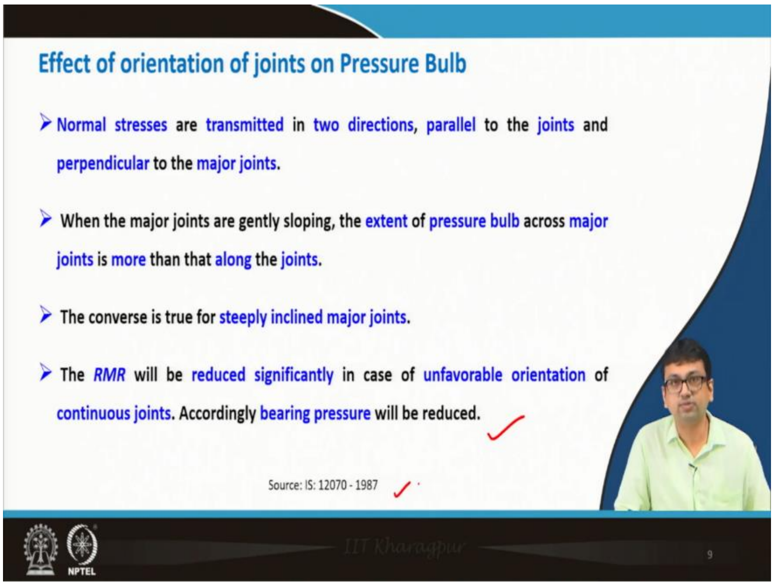

Now, another thing we should discuss also that is the effect of orientation of joints on the pressure bulb. I have shown you some diagrams in the previous lecture. So, we can discuss more about this like the normal stresses are transmitted in two directions, parallel to the joints and perpendicular to the major joints.

Then when the major joints are gently sloping, the extent of pressure bulb across major joints is more than that along the joints. The converse is true for steeply inclined major joints and finally, the RMR will be reduced significantly in the case of unfavourable orientation of continuous joints accordingly bearing pressure will be reduced. So, these are some of the general comments related to the effect of orientation of joints on pressure bulb provided in IS-12070.

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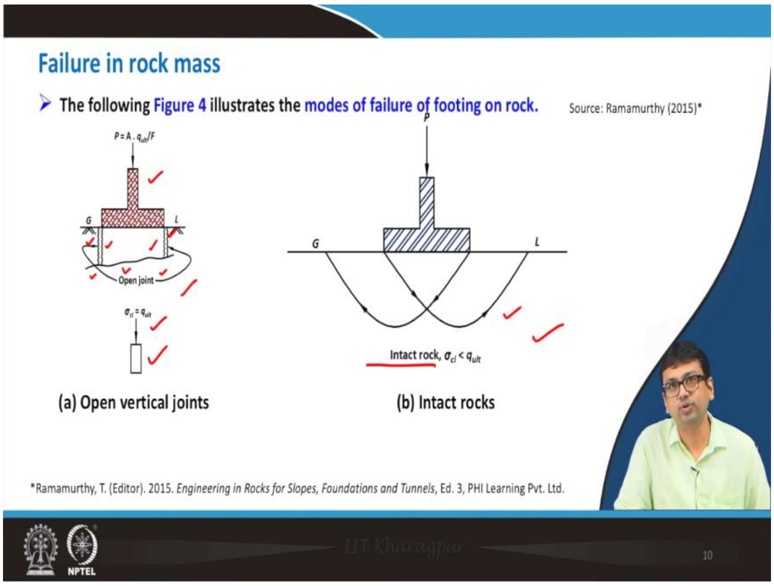

Now, we will discuss about the different modes of failures in rock mass. These are some of the figures which will illustrate the modes of failure of footing on rock. So, first we can see the open vertical joint.

If the foundation is like this, the failure will occur through this which is clearly understandable. Now, for intact rock, this type of failure pattern will develop. And, we are familiar with this type of failure pattern in soil mechanics.

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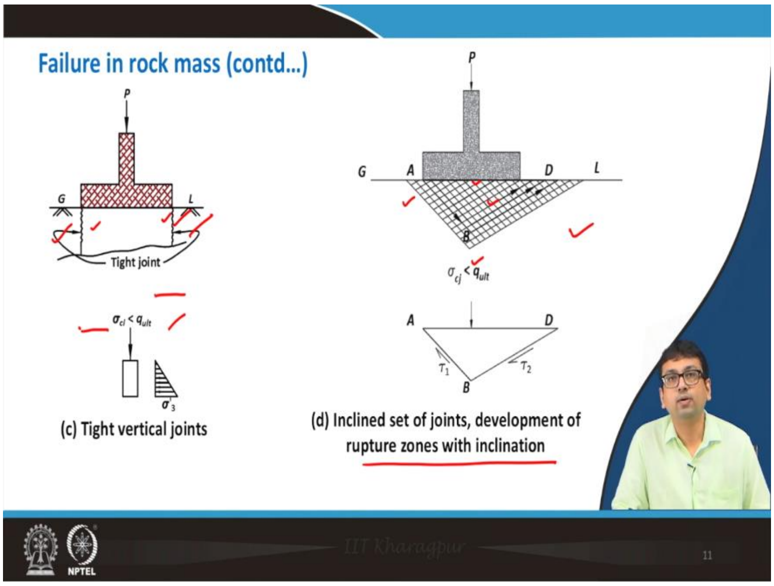

Now, the previous figure illustrated the open joint. This figure illustrates the tight vertical joints, i.e., there is no gap. And, if there is the inclined set of joints, you can see the development of rupture zone with inclination.

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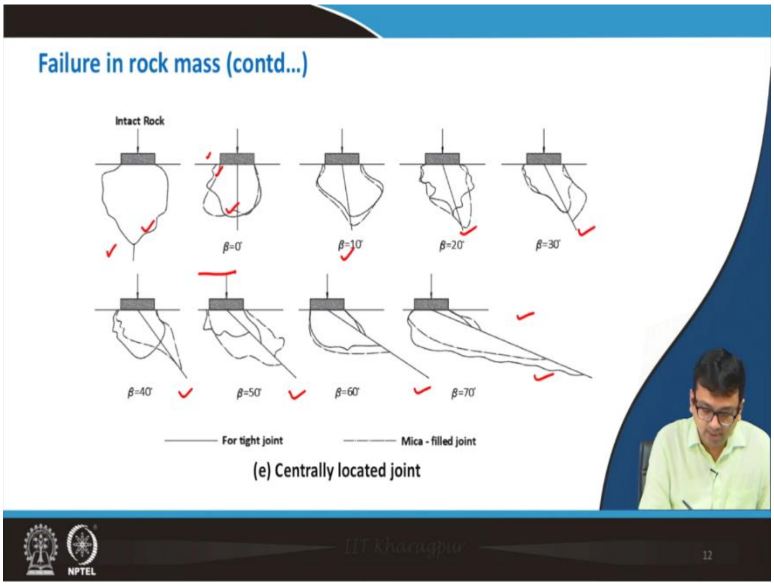

Also, like intact rock is here and in that case, centrally located joint suppose here you see the joint is like this. So, then this type of failure pattern you may observe and if it is you see directly vertical. So, the angle β is equal to 0° so, this type of modes of failure you can observe; then 10°; then 20°, then 30°, 40°, 50°, 60°, 70°, you can see over here. So, how the mode the failure modes are changing with the change in angle β.

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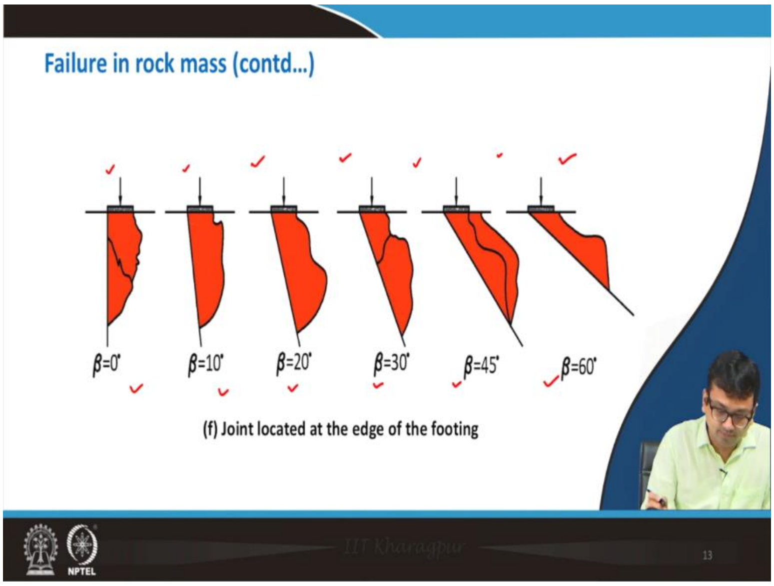

Now, this figure illustrates the joint located at the edge of the footing. So, the joint is located at the edge of the footing. So, from the edge of the footing, the orientation angle (β) is changing. So, here it was 0º, 10º, 20º, 30º, 45º, 60º and these type of failure patterns, you can observe over here.

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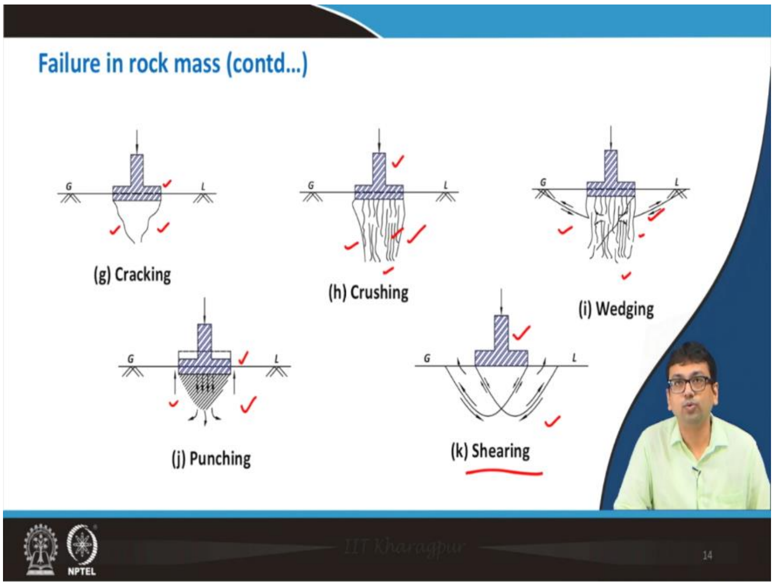

Now, if there is simple cracking, then the foundation will just come down and this type of cracks has developed. Now crushing you see you can look at over here, here, here, here. The rock mass has crushed when the footing has that has gone down. The failure has occurred and the rock must has crushed. So, we can observe this type of failure mode.

The next one is the wedging. You can see over here, the wedges are developing and the crushing is also observed in this part. Then punching, so it is you can see simply it has just punched, i.e., the punching failure has occurred. The next one is the shearing with which we are very familiar. Like in the case of soils also, we can see this type of shearing failure. So, the failure modes are cracking, crushing, wedging, punching and shearing.

So, shearing failure is the most desirable and in the case of other failures, these are failing suddenly. So, sudden crack may develop, and if the rock masses is very poor, crushing may occur; then this is the wedging but some small-small mass cracks are visible in the rock mass (i.e., crushing), and you can see here the punching. Punching can be dangerous as it may cause a sudden failure. It is the shearing failure which is the most desirable mode of failure and we are familiar with this type of failure. Anyway, so, I wanted to show you some different modes of foundation failure. So, I think you have understood from these different diagrams.

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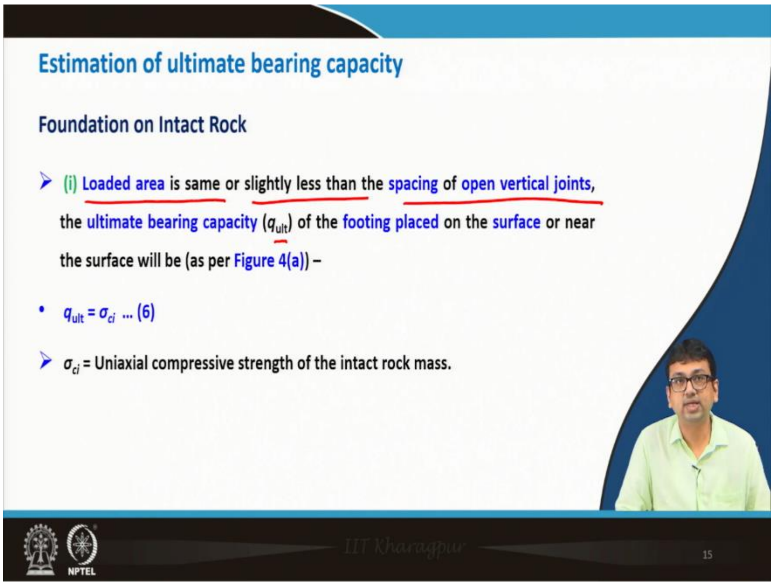

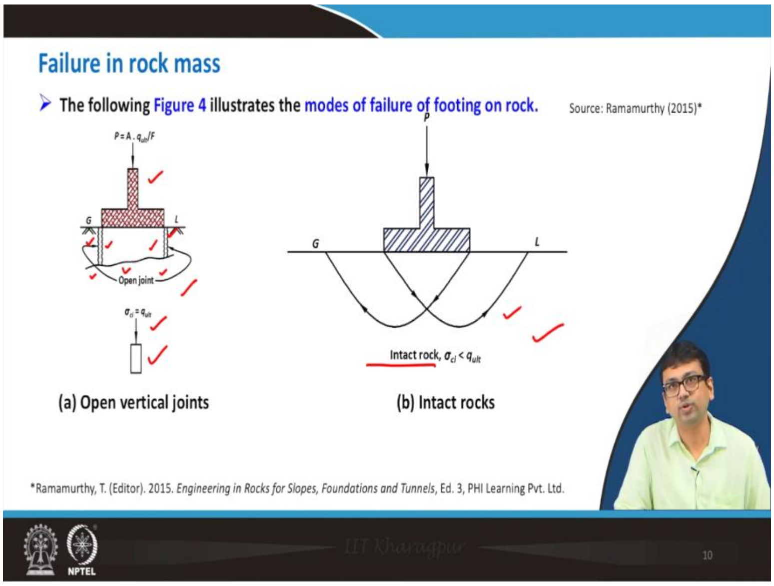

Now related to these diagrams also, at the first let us see what is written over here and then I will go to the corresponding figure. Foundation on intact rock suppose. So, (i) the loaded area is same or slightly less than the spacing of open vertical joints. The ultimate bearing capacity, q_{ult} ('*ult*' is for ultimate) of the footing placed on the surface or near the surface will be (as per figure 4a), $q_{ult} = \sigma_{ci}$.

Now σ_{ci} is the uniaxial compressive strength of the intact rock mass. So, from the figure 4a, we can see that there are gaps between the joints, the spacing of joints is either equal to the footing width or more than the footing width. So, in that case obviously, what will happen? And, there is open joint also. So, it is similar to the condition of uniaxial compressive strength test.

So, you can see the open joint and the spacing of joints is equal to or maybe likely more than the footing width. In that case, it is very much similar to uniaxial compressive strength test. So, q_{ult} ultimately will be equal to σ_{ci}.

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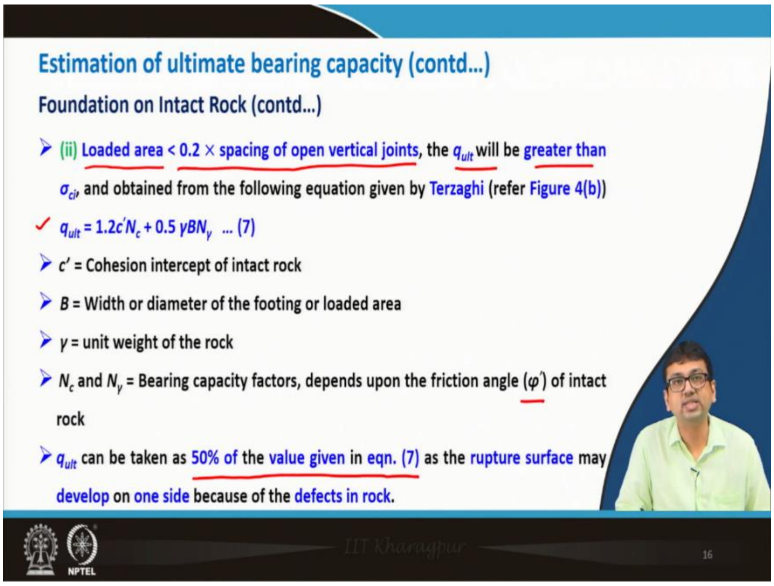

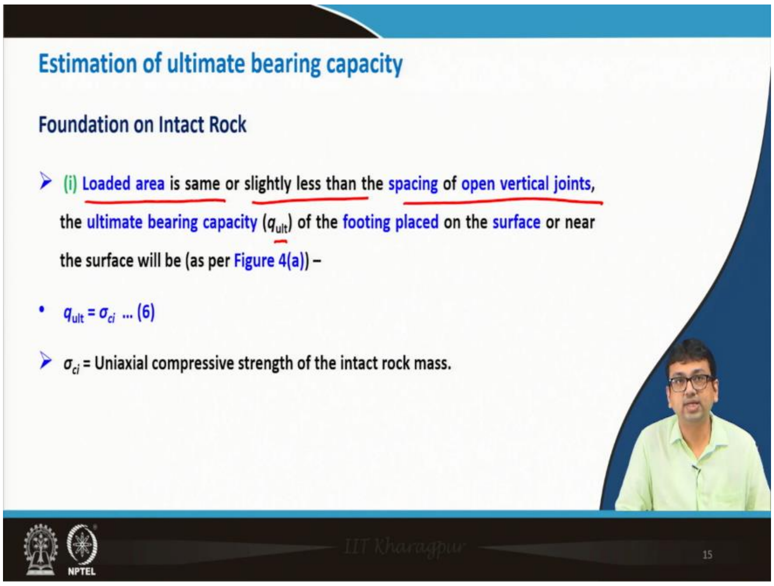

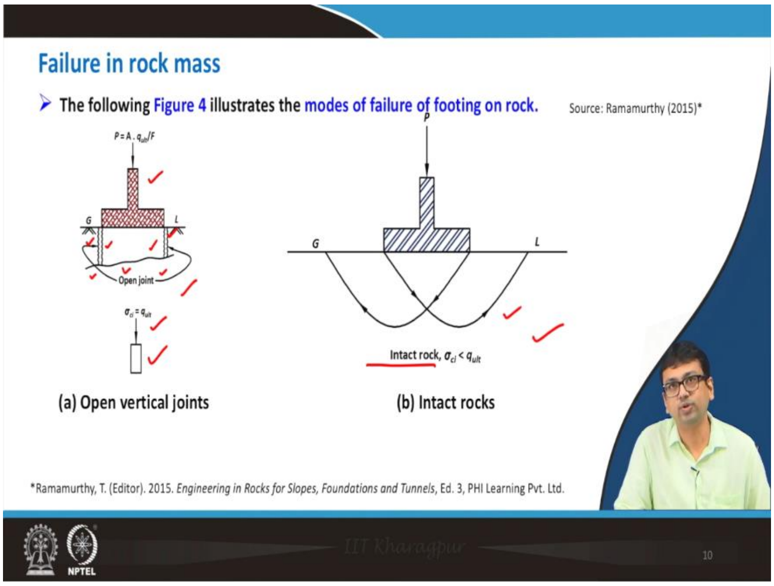

When, the loaded area is < 0.2 × spacing of open vertical joints. So, loaded area is < 0.2 × spacing of open vertical joints, the q_{ult} will be greater than σ_{ci}. In the previous case, the loaded area is same or slightly less than the spacing. Now, in this case, the loaded area is < 0.2 × the spacing of the open vertical, the q_{ult} will be greater than the σ_{ci} and obtained from the following equation given by Terzaghi so, $q_{ult} = 1.2c' N_c + 0.5\gamma B N_\gamma$.

Now, c' is the cohesion intercept of intact rock and B is the width or the diameter of the footing or loaded area and γ is the unit weight of the rock, and N_c and N_γ are the bearing factors depends upon the friction angle (φ') of intact rock. Now, it also says that q_{ult} can be taken as 50% of the value given in Eq. (7) as the rupture surface may develop on one side because of the defects in rock. So, in that case, it is recommended to take 50% of the value given by Eq. (7).

Now, as let us just quickly see the Figure 4b. So, basically loaded area is < 0.2 × spacing of the vertical joints. So, loaded area is much smaller than the spacing between the joints or the distance between the joints.

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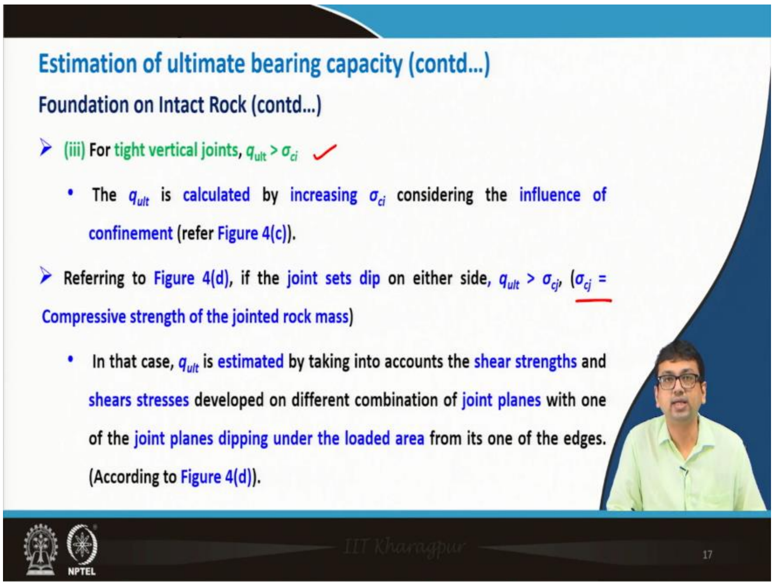

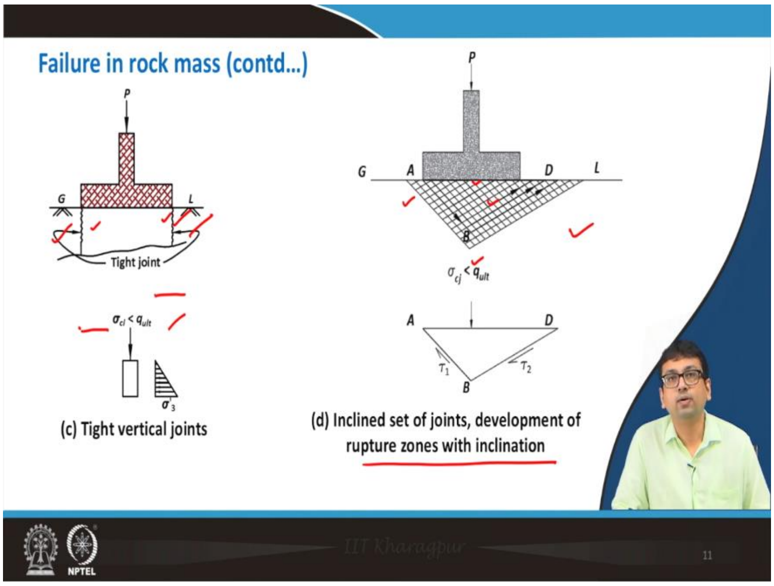

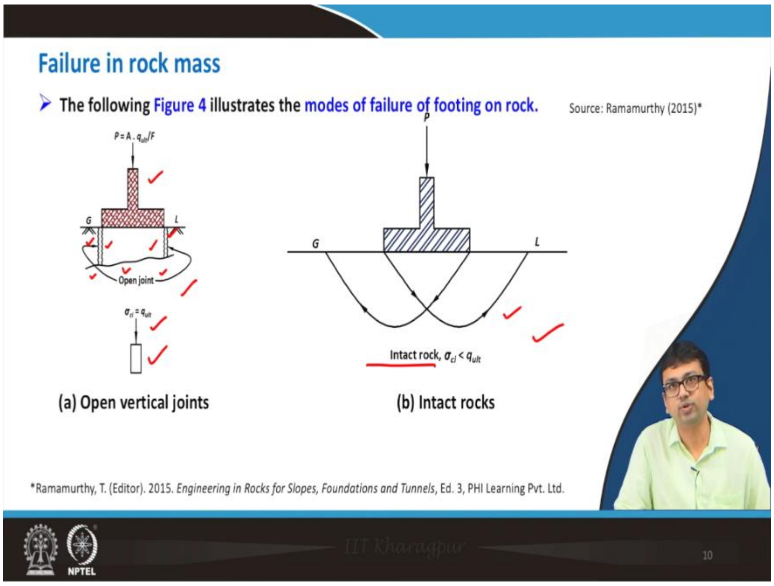

Now, for the foundation on intact rock, the third condition is for tight vertical joints, q_{ult} should be greater than σ_{ci}. The q_{ult} is calculated by increasing σ_{ci} considering the influence of confinement. So, that is the Figure 4c where the tight joints are present. So, the q_{ult} will be greater than σ_{ci} because of the confining effect, whereas if it is an open joint, there will be no confinement but in this case, confinement effect will come. So, now, referring to figure 4d, if the joint sets dip on either side, q_{ult} will be greater than σ_{cj}.

Now σ_{cj} is the compressive strength of the jointed rock mass. In that case, q_{ult} is estimated by taking into account the shear strengths and the shear stresses developed on different combination of joint planes with one of the joint planes dipping under the loaded area from its one of the edges. So, this is according to figure 4d, the inclined set of joints, and development of ruptured zones with inclination.

So, in that case, the guideline is like that as I have discussed. So, q_{ult} is greater than σ_{cj}, here σ_{cj} is the uniaxial compressive strength. So, σ_{ci} is the uniaxial compressive strength of the intact rock, whereas σ_{cj} is the compressive strength of the jointed rock mass.

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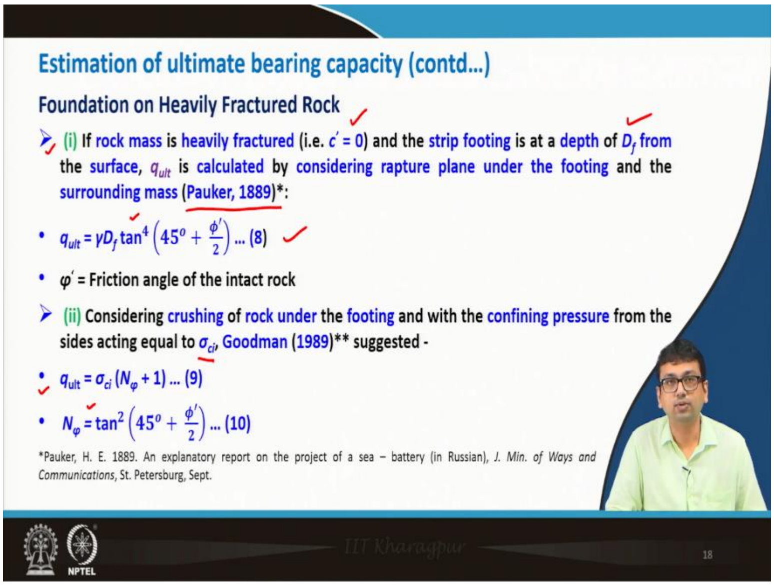

Now, foundation on heavily fractured rock. If the rock mass is heavily fractured (i.e., $c' = 0$) and the strip footing is at a depth of D_f from the surface, then q_{ult} is calculated by considering that rupture plane under the footing and the surrounding mass. So, as per Pauker (1889), $q_{ult} = \gamma D_f \tan^4\left(45^o + \frac{\varphi'}{2}\right)$, where φ' is the friction angle of the intact rock.

Now, this is the first condition where the rock mass is heavily fractured and the strip footing is at a depth of D_f from the surface. Considering crushing of rock under the footing with the confining pressure from the sides acting equal to σ_{ci}, Goodman (1989) suggested $q_{ult} = \sigma_{ci}(N_\varphi + 1)$, where $N_\varphi = \tan^2\left(45^o + \frac{\varphi'}{2}\right)$.

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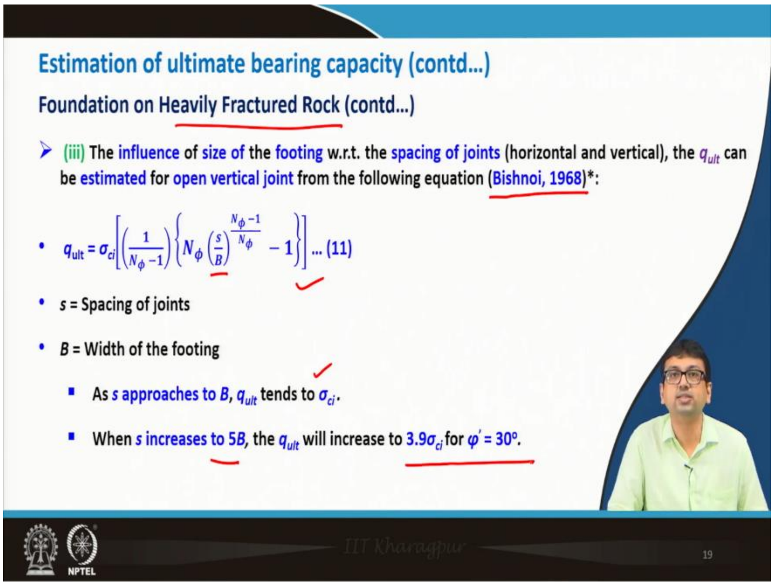

Now, we are continuing with the foundation on heavily fracture rock. So, third is the influence of size of the footing with respect to the spacing of joints horizontal and vertical, the q_{ult} can be estimated for open vertical joints from the following equation given by Bishnoi (1968). So, here we can see, $q_{ult} = \sigma_{ci}\left[\left(\frac{1}{N_\varphi - 1}\right)\left(N_\varphi\left(\frac{s}{B}\right)^{\frac{N_\varphi - 1}{N_\varphi}} - 1\right)\right]$.

Here, in the equation, s is the spacing of joints, B is the width of footing. So, (s/B) will become a non-dimensional parameter. As s approaches to B, q_{ult} tends to σ_{ci}. So, as the s increases to 5B, the q_{ult} will increase to 3.9σ_{ci} for φ = 30º. So, we can get some guidelines from Bishnoi in 1968.

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So, okay let us conclude here today. Thank you.