Rock Mechanics and Tunneling
Professor Debarghya Chakraborty
Department of Civil Engineering
Indian Institute of Technology Kharagpur
Lecture 31
Introduction to Rock and Rock Mass Failure

Hello everyone! I welcome all of you to the 5th lecture of Module 6. In Module 6, we are discussing the rock and rock mass failure criteria. In the next module (i.e., module 7) also, we will continue the discussion related to rock and rock mass failure criteria. So, in this week, we have primarily discussed the analysis of stresses, stress-strain relationships, and etcetera. And today, we will discuss the introduction to rock and rock mass failure.

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So, we will discuss the types of fracture, triaxial strength of rock, effects of confining stress, and the different models for rock structure.

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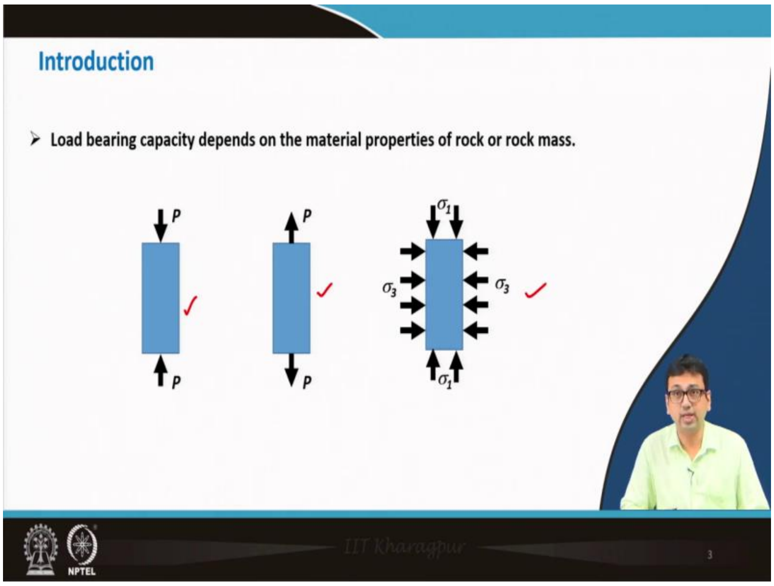

We know that the load bearing capacity depends on the material properties of rock or rock mass. We can see that the first figure represents the uniaxial compression, the second figure represents the natural tension, and the last one represents the triaxial compression. Now, the behavior of the rock or rock mass depends on its material properties.

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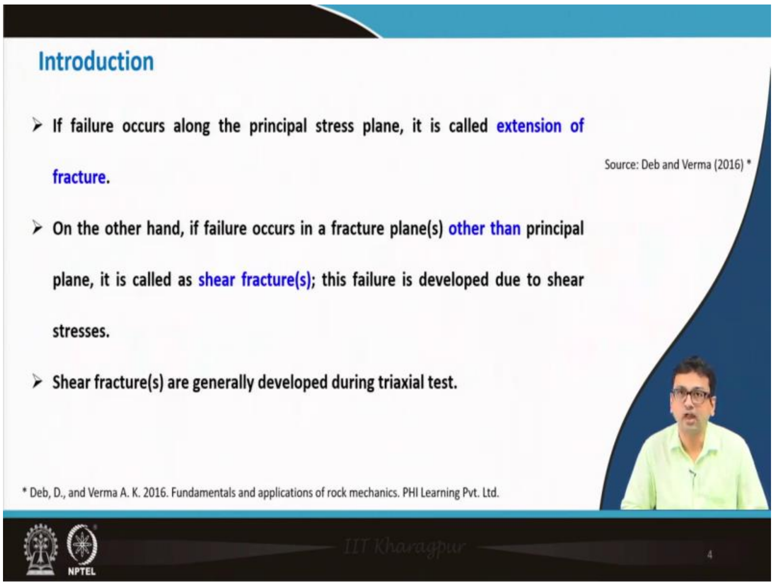

It is stated that if the failure occurs along the principal stress plane, it is called extension of fracture. On the other hand, if failure occurs in a fracture plane(s) other than principal plane, it is called as shear fracture(s). This failure is developed due to the shear stresses and the shear fracture(s) are generally developed during the triaxial test.

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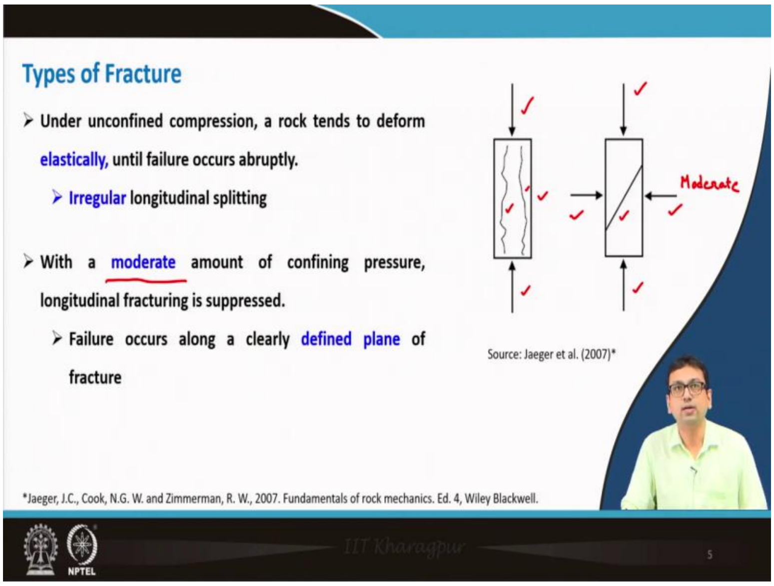

Now, let us discuss the types of fracture. So, we can see over here that under the unconfined compression (i.e., only the axial compression is applied, no confinement is there), a rock tends to deform elastically, until failure occurs abruptly, and the irregular longitudinal splitting can be observed.

Now, in the second diagram, the confining stress is also applied along with axial compression. So, with a moderate amount of confining pressure, longitudinal fracturing is suppressed, and the failure occurs along a clearly defined plane of fracture.

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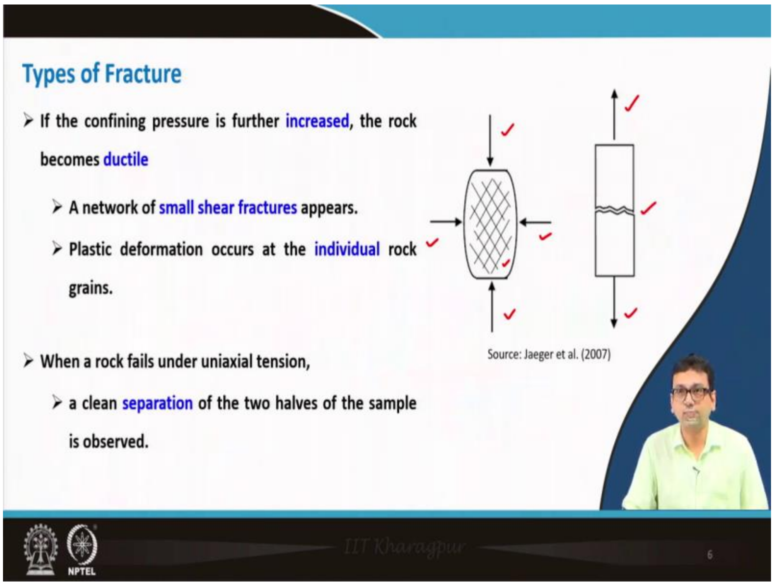

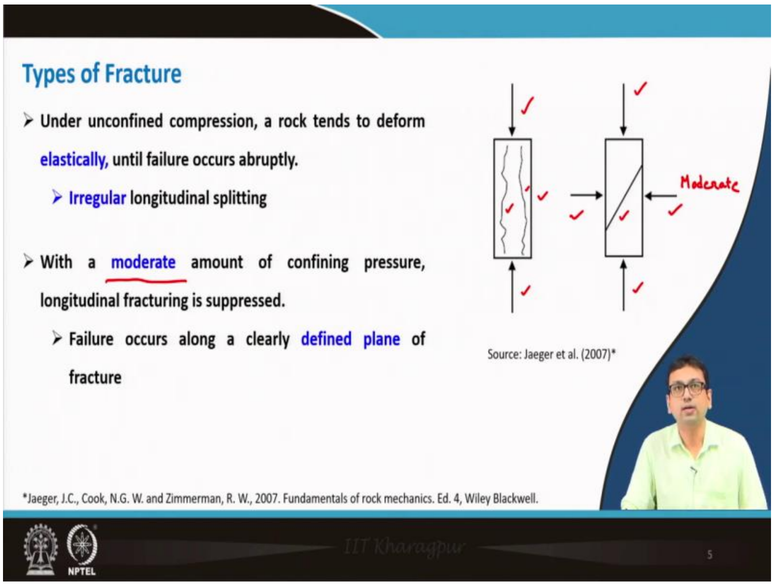

Now another case is that if the confining pressure is further increased (i.e., the axial load is applied along with the increasing confining pressure), the rock becomes ductile. The network of small shear fractures appears. As we can see that the network of small shear fractures develops and the plastic deformation occurs at the individual rock grains.

The last figure shows that when a rock fails under uniaxial tension (i.e., no confining pressure is there), clear separation of the two halves of the sample is observed. So, these are the different types of fracture.

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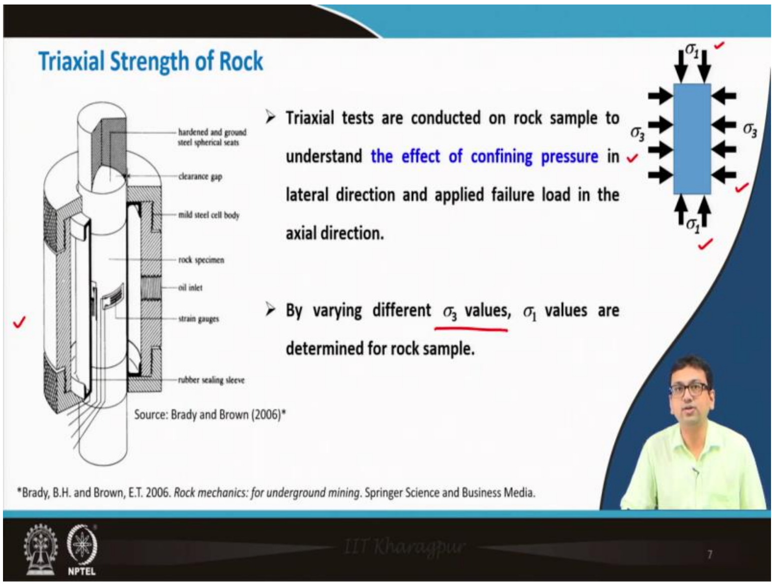

Now, we will discuss the triaxial strength of rock. We have discussed in detail about the triaxial compression test and there we have learnt about the Mohr circle and how to find out the c–ϕ parameters. We have seen the diagram of the triaxial cell for the testing of rock, and in the right hand side diagram, we can see that σ_1 is the major principle stress (i.e., the axial compression) and σ_3 is the minor principle stress.

So, here the confining pressure is σ_3. So, the triaxial tests are conducted on rock sample to understand the effect of confining pressure in lateral direction and applied failure load in the axial direction. By varying these σ_3 values, σ_1 values are determined for the rock sample.

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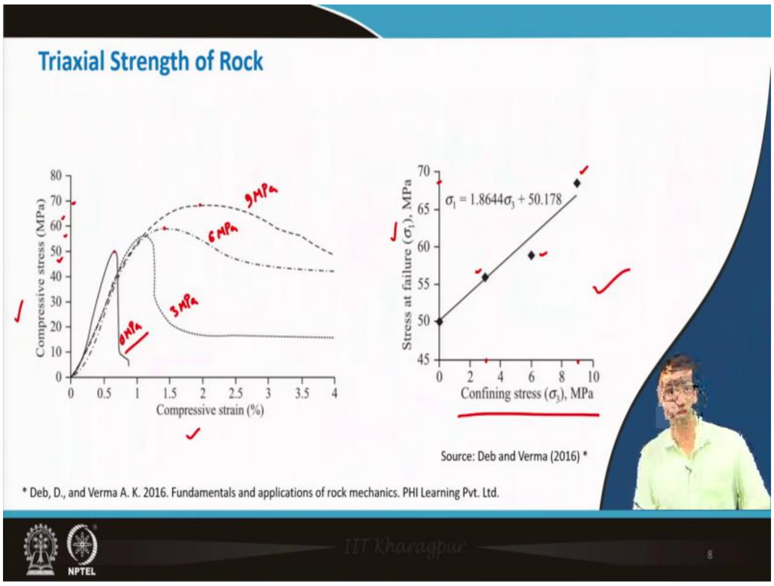

Now, let us see a typical plot for compressive stress versus compressive strain. There are total four curves where the first one is for 0 MPa, the second one is for 3 MPa confining pressure, the third one is for 6 MPa, and the last one for 9 MPa confining pressure.

For the first curve, the peak is found to be around 50 MPa corresponding to 0 MPa confining pressure which represents mainly the uniaxial compressive strength test. So, the failure is occurring, i.e., the ultimate load carrying capacity is around 50 MPa.

Whereas for 3 MPa confining pressure, the peak is around 55 MPa for the triaxial test. Then, for 6 MPa, it is around 60 MPa, and for the 9 MPa confining pressure, it is about 70 MPa.

So, in the right hand side diagram, we can see that the horizontal axis represents the confining stress, whereas the vertical axis represents the stress at failure. So, the stress at failure is the σ_1, and the confining stress is the σ_3. So, when, the σ_3 is 0, i.e., when we are performing the unconfined compressive strength test, the σ_1 is becoming about 50 MPa.

The σ_1 is about 55 MPa corresponding to the 3 MPa confining pressure. Then corresponding to 6 MPa confining stress (σ_3), the σ_1 is around 60 MPa, and then corresponding to 9 MPa confining stress, the σ_1 is around 70 MPa.

Now, let us try to get few more information from these two diagrams.

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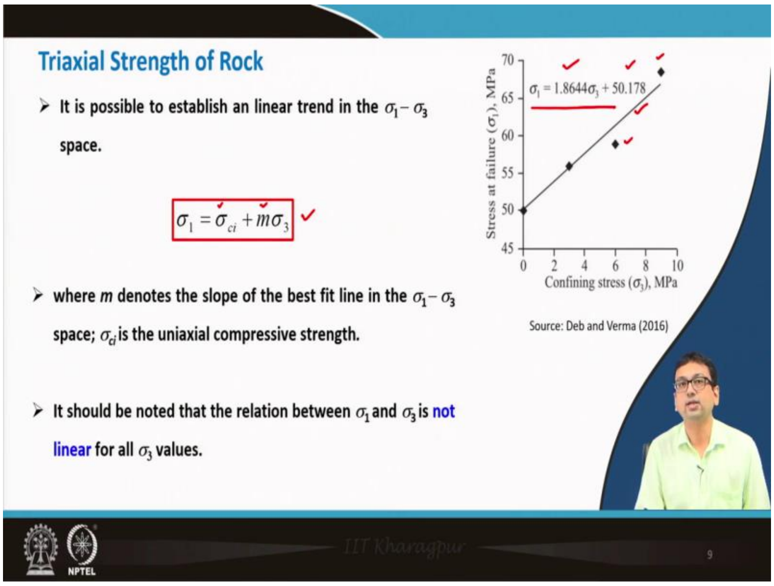

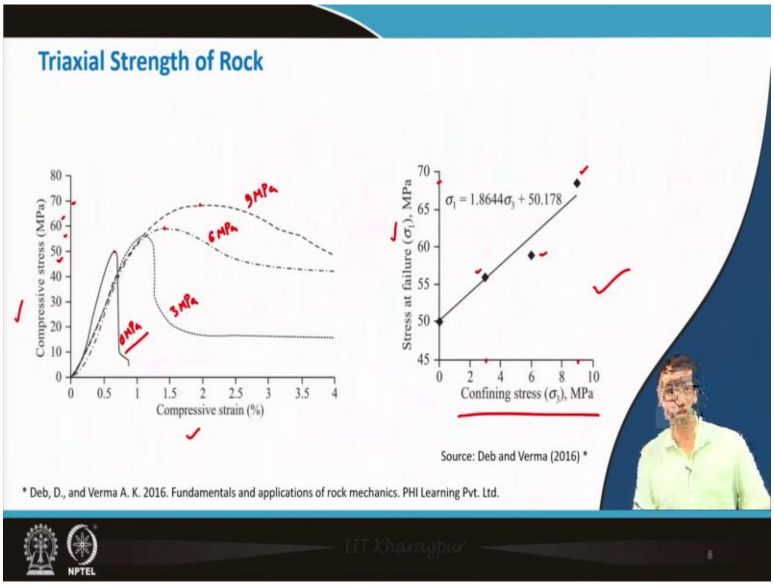

Now, if we focus on the second diagram, we can say that it is possible to establish a linear trend between σ_1 and σ_3. So, this is the linear trend what is fitted over here and we can get the equation like this, which is shown over here (i.e., $\sigma_1 = 1.8644\sigma_3 + 50.178$). From this equation, we can get the uniaxial compressive strength (σ_{ci}) and the m denotes the slope of the best fit line in the σ_1 - σ_3 space.

From the right hand side diagram, we can see that σ_{ci} is 50.178 MPa as the best fit plot is cutting the σ_1-axis at 50.178, and the slope of the best fit curve (m) is 1.8644.

However, it should be noted that the relationship between σ_1 and σ_3 is not linear as for all σ_3 values the corresponding σ_1 values are not exactly falling on the straight line. Hence, the relationship is not completely linear. Though the best fit curve is found to be a linear line and the corresponding equation is $\sigma_1 = \sigma_{ci} + m\sigma_3$, where m = 1.8644 and σ_{ci} = 50.178 MPas.

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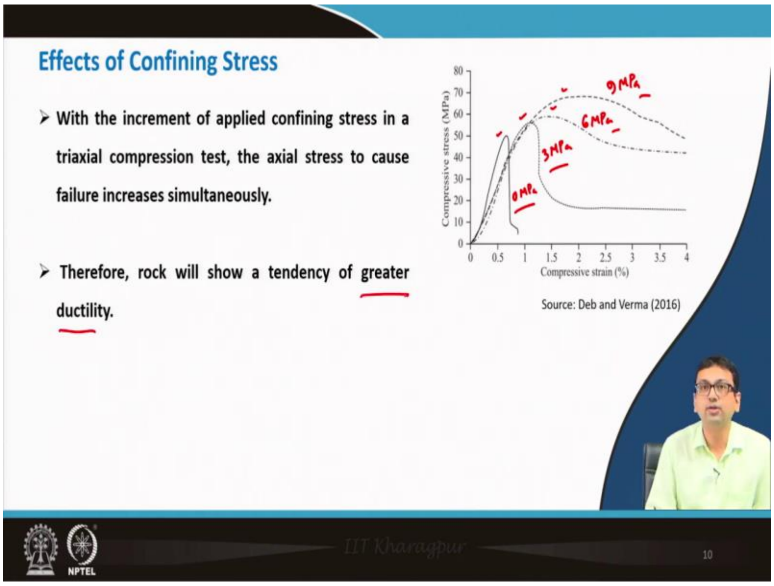

Now, we will discuss more about the effects of confining stress. So, we can see from the diagram that with the increase in applied confining stress in the triaxial compression test, the axial stress to cause failure increases simultaneously. So, the peak of the curve is increasing continuously with the increase in confining pressure. Therefore, the rock will show a tendency of greater ductility with the increasing confining pressure.

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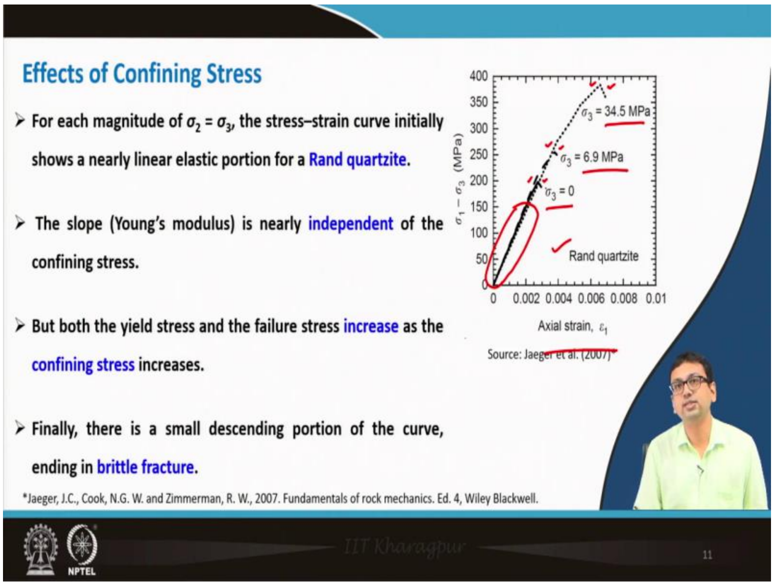

Now, let us continue the discussion on the effect of the confining stress only. Now, we will into the plot of $(\sigma_1 - \sigma_3)$ versus axial strain for Rand quartzite. Since for the triaxial test, $\sigma_2 = \sigma_3$. Hence, for each magnitude of $\sigma_2 = \sigma_3$, the stress-strain curve initially shows a nearly linear elastic portion for a Rand quartzite.

We can see that whether the σ_3 = 0 MPa or σ_3 = 6.9 MPa or σ_3 = 34.5 MPa, the initial portion of the curve is almost linear. The slope is the Young's modulus, which is nearly independent of the confining stress.

But, both the yield stress and the failure stress increase as the confining stress increases. Finally, there is a small descending portion of the curve indicating the brittle fracture.

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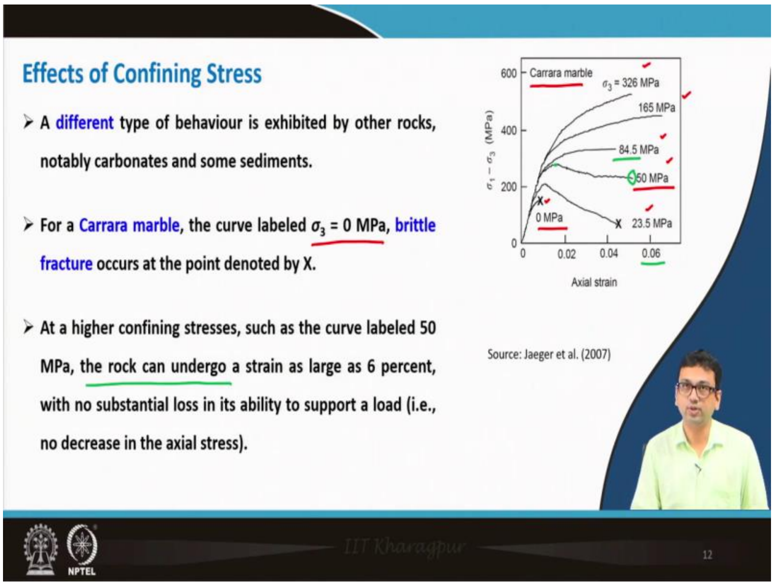

Now, let us see the $(\sigma_1 - \sigma_3)$ versus axial strain plot for the Carrara marble. A different type of behavior is exhibited by other rocks, notably carbonates and some sediments. The curve labeled as σ_3 = 0 MPa indicates the unconfined compressive strength test. For a Carrara marble, the curve labeled as σ_3 = 0 MPa, brittle fracture occurs at the point denoted by X.

Now at higher confining stresses, i.e., the stresses are like 23.5, 50, 84.5, 165 and 326 MPa. Now, if we consider the confining stress (σ_3) as 50 MPa case, the rock can undergo a strain as large as 6 percent with no substantial loss in its ability to support a load, i.e., there will be no decrease in the axial stress. However, there is a little dip with respect to the ultimate point, but there is no substantial decrease in axial stress, whereas for σ_3 = 84.5 MPa, the curve is almost flat.

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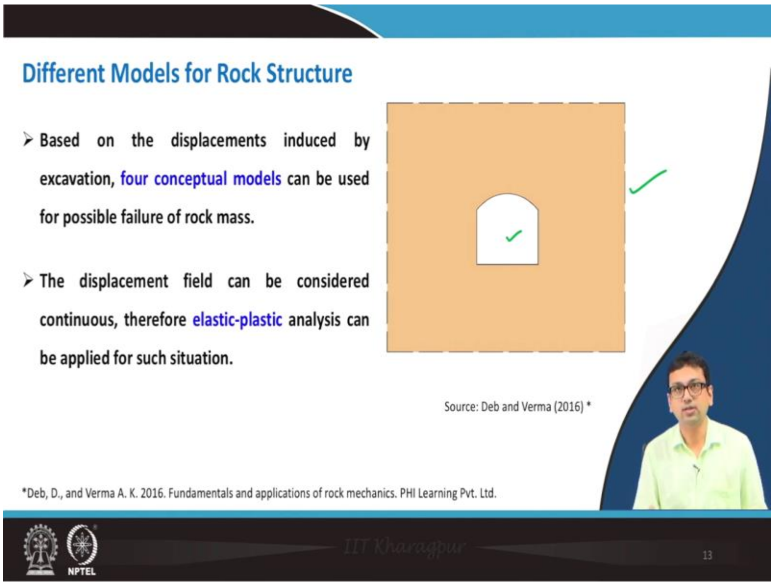

Now, we will learn the different models for rock structure. So, based on the displacements induced by excavation, four conceptual models can be used for possible failure of the rock mass. So what are they? As we can see over here that the diagram represents an ideal case where no joints are present over here. It is a completely continuous rock mass. So, the displacement field in this case can be considered continuous therefore, elastic-plastic analysis can be applied for such situation. It is considered to be the most desirable condition.

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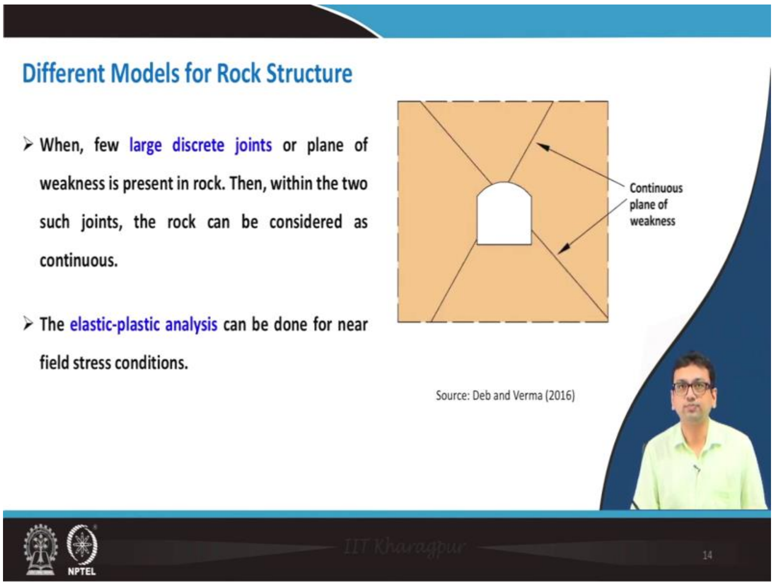

When, few large discrete joints or plane of weakness is present in rock, within the two such joints, the rock can be considered as continuous. So, we can see in the diagram that the rock mass can be considered as continuous in between these joints here. So, within two such joints, the rock can be considered as continuous and the elastic-plastic analysis can be done for near field stress conditions.

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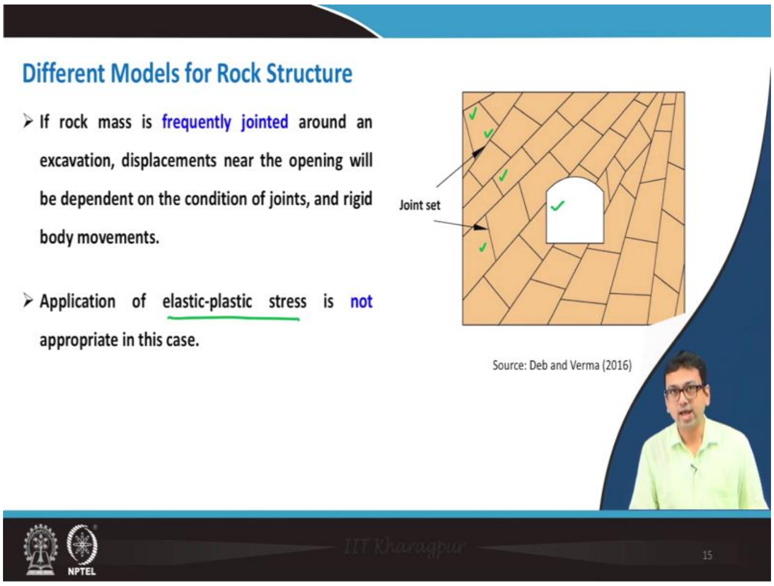

In this diagram, we can see that the rock is frequently jointed. So, the rock mass is frequently jointed as compared to the previous one; several joint sets are there.

So, if a rock mass is frequently jointed around an excavation, displacements near the opening will be dependent on the condition of joints and rigid body movements. Rigid body movement means the translation and rotation. Application of elastic-plastic stress is not appropriate in this case.

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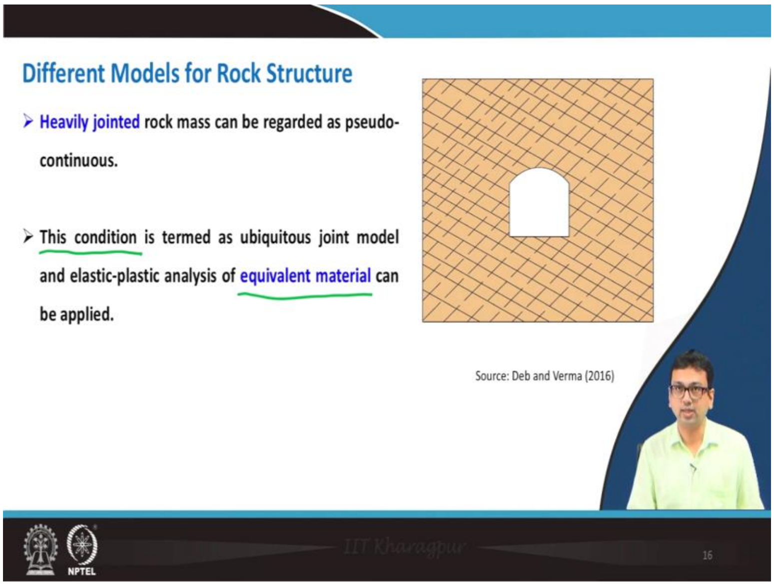

Now, another case represents the heavily jointed rock mass. The heavily jointed rock mass can be regarded as pseudo continuous. This condition is termed as ubiquitous joint model, and the elastic-plastic analysis of equivalent material can be applied.

So, we can apply the elastic plastic analysis for this case, but we have to find out the equivalent material parameters, like the equivalent strength the rock mass. Then, based on that, we can go for the elastic plastic analysis, if we want to find out the stresses developing around this excavation.

So, how much stress is developing in the surrounding region, the equivalent material is needed to be considered to find out that in a simplified manner. Then, we can have to go for this elastic-plastic analysis.

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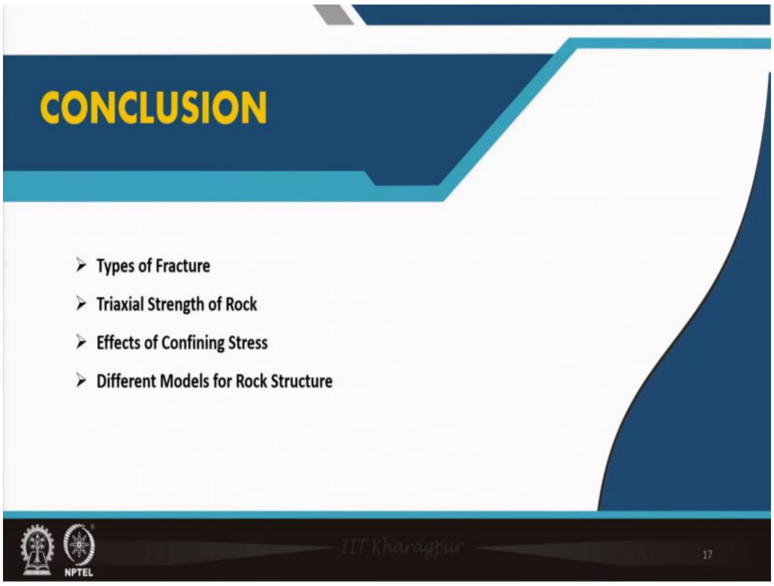

So, in the today's lecture, we have discussed about the types of fracture, triaxial strength of rock, effects of confining stress, and finally different models for rock structure. In the next module, we will continue with the rock and rock mass failure criteria, and we will try to discuss about the different rock mass failure criteria.

We will mainly focus on the Hoek-Brown yield criteria, other than that we all will also learn about the Mohr-Coulomb yield criteria, Drucker-Prager yield criteria and several other. These yield criteria are commonly used in rock mass modeling. So, thank you. Let us conclude here today.