Retrofitting and Rehabilitation of Civil Infrastructure
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Lecture 33
Design Approach for Flexural Strengthening

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Hello friends, welcome to the NPTEL online certification course, Retrofitting and Rehabilitation of Civil Infrastructure. Today we will discuss module E, the topic for module E is retrofitting using fiber reinforced polymer composites.

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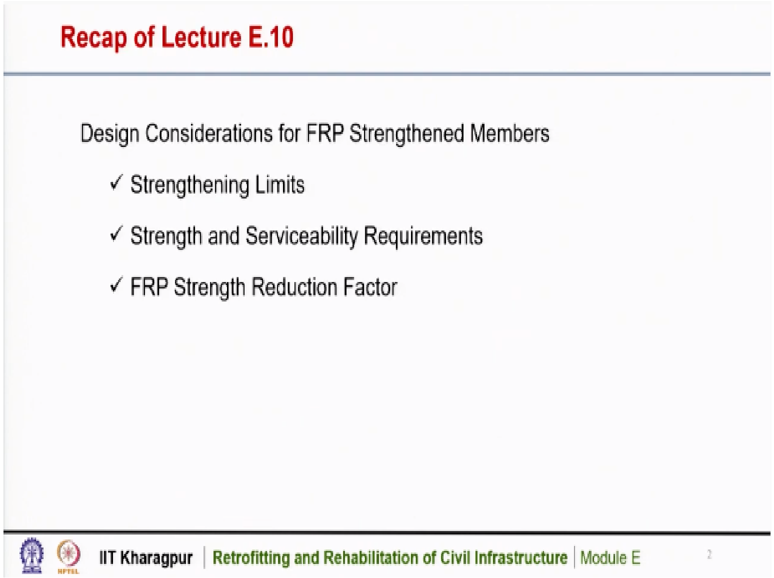

In the previous lecture we have discussed the design considerations for FRP strengthened members. We have discussed the strengthening limits, the strength and serviceability requirements and FRP strength reduction factor for the design of FRP strengthened members.

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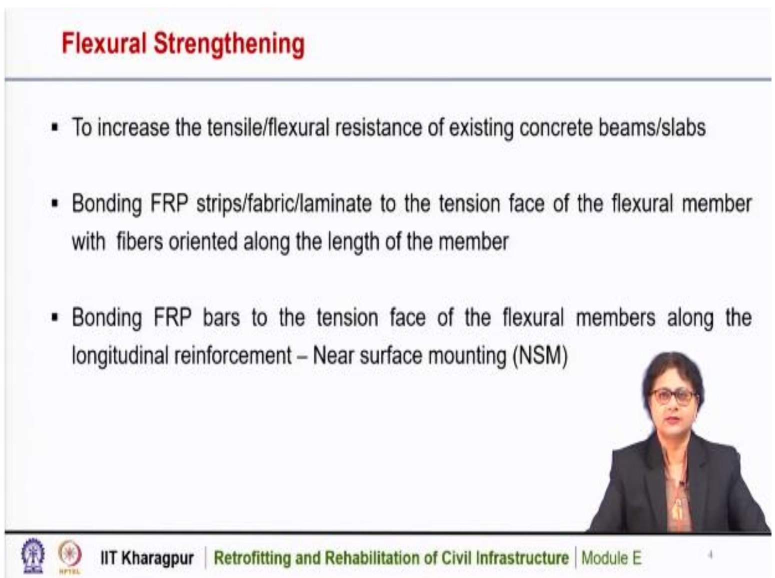

Today we will discuss the design approach for FRP retrofitted flexural member, we have discussed earlier that flexural strengthening of existing members is aimed to increase the tensile or flexural resistance of existing concrete structures and for this purpose FRP strips or fabrics or laminates are bonded to the tension phase of the flexural member with fibers oriented along the length of the member.

The flexural strengthening can also be done using near surface mounting reinforcement, here the FRP bars are bonded to the tension phase of the flexural members along the longitudinal reinforcement. So, in both cases whether it is near surface mounted reinforcement or externally bonded FRP strips or fabric or laminate, the tensile capacity of the member is increased significantly.

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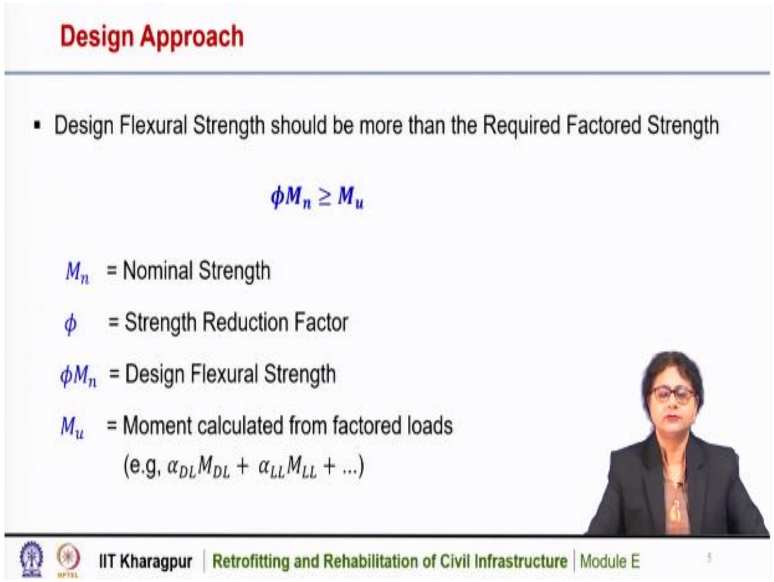

Now, for the design of the flexural strengthening of an existing member using fiber reinforced polymer composite, the design is based on limit state method and the design flexural strength should be more than the required factored strength.

So, the design flexural strength it should be more than the required factored strength, that is $\boldsymbol{\phi M_n} \geq \boldsymbol{M_u}$, where $\boldsymbol{\phi M_n}$ is the design flexural strength or moment, M_n is the nominal strength and $\boldsymbol{\phi}$ is a strength reduction factor. M_u is the moment calculated from factored loads and this may be due to dead load, life load etc. So, the design flexural strength that is the $\boldsymbol{\phi M_n}$ should be more than the required strength that is M_u.

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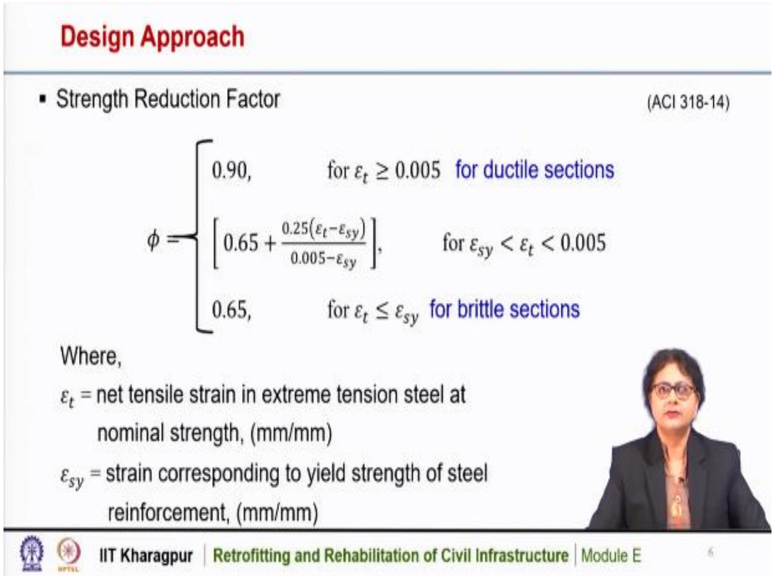

The strength reduction factor may be different depending on the type of section, for ductile sections the ϕ can be taken as 0.9 when the ε_t, that is the net tensile strain in the extreme tension steel at nominal strength is more than equal to 0.005 that is for ductile sections. And the ϕ can be taken as 0.65 for brittle sections when the ε_t is less than equal to ε_{sy} that is the strain corresponding to yield strength of steel reinforcement and ϕ can be taken as the intermediate value when the strain levels are in between these two ranges. So, these are the range of values of ϕ that can be used to calculate the design strength.

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Now, we will discuss the design approach for FRP flexural strengthening and this has been done by considering strain compatibility, internal force equilibrium and possible modes of failure. There are different guidelines available American Concrete Institute guideline or British Standard Guidelines or Australian Standard Guidelines. However, in all these guidelines the basic design approach is the same that considers the strain compatibility, the internal force equilibrium and the possible modes of failure of the FRP retrofitted flexural members.

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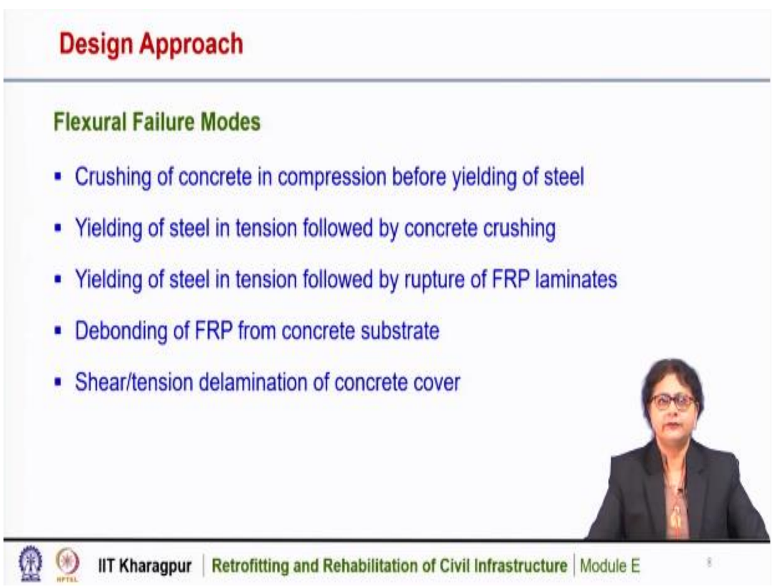

The possible flexural failure modes have been considered and these have been developed from the various research works on flexural members retrofitted with FRP. The possible flexural failure modes are crushing of concrete in compression before yielding of the steel, it could be yielding of steel in tension followed by concrete crashing or it could be yielding of steel intention followed by rupture of FRP laminates. Debonding of FRP can also be there from the concrete substrate or shear or tension delamination of the concrete cover.

So, these are the possible failure modes that has been observed from various research works and, in the design, also we will consider these failure modes and the design should be done so that these failure modes can be avoided.

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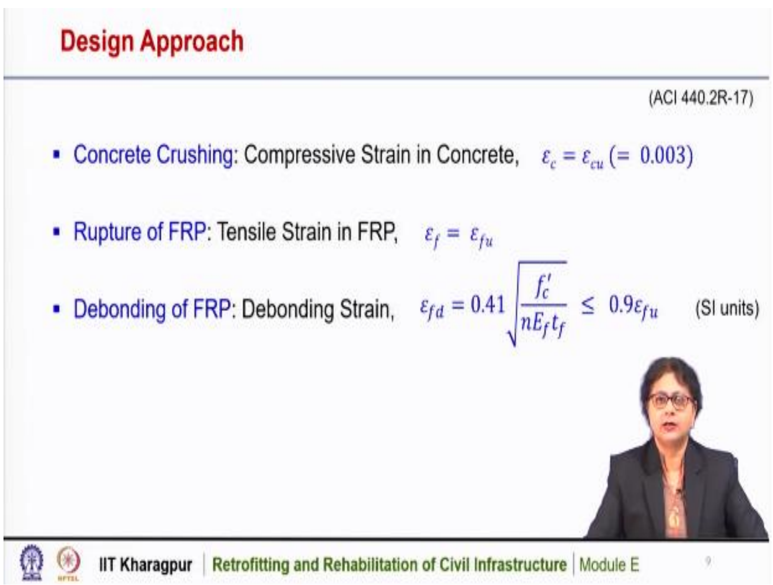

Now, the failure could be due to concrete crushing and this concrete crushing may occur when the compressive strain in concrete reaches its ultimate strain, that is the compressive strain in concrete is ε_c when it reaches the ultimate strain of concrete, which is denoted as ε_{cu} then there may be concrete crashing. And that ultimate strain is generally taken as 0.003.

Failure could be due to rupture of FRP, the rupture of FRP can occur when the tensile strain in FRP reaches the ultimate strain of the FRP, that is ε_f that is the strain developed in the FRP reaches the ultimate strain of the FRP material that is ε_{fu}.

So, when the strain in the FRP reaches the ultimate limiting value then there may be rupture of the FRP. Failure is also due to debonding of the FRP from the concrete substrate and this debonding can occur when the debonding strain that is $\varepsilon_{fd} = 0.41\sqrt{(f_c'/nE_f t_f)} \leq 0.9\ \varepsilon_{fu}$ that is the ultimate strain of FRP and this expression is in SI units. So, these are the conditions for concrete crushing or rupture of FRP or debonding of the FRP that has been considered in ACI.

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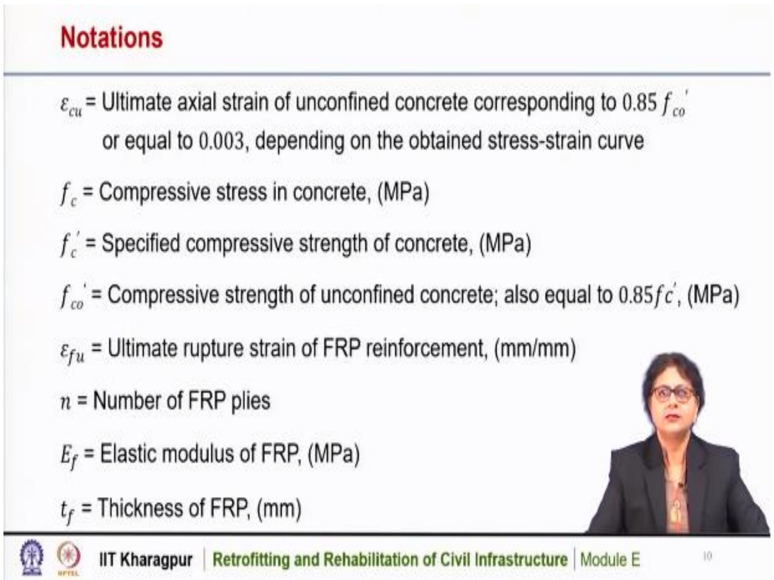

And these are the notations and these notations have been used for further discussion also epsilon cu is the ultimate axial strain of unconfined concrete and that is equal to 0.003 and that corresponds to 0.85 f_{co}', f_{co}' is the compressive strength of unconfined concrete, f_c is the compressive strength of concrete, f_c' is the specified compressive strength of concrete, ε_{fu} is the ultimate rupture strain in FRP, n is the number of FRP plies, E_f is the elastic modulus of the FRP and t_f is the thickness of the FRP. So, these are the notations that are used.

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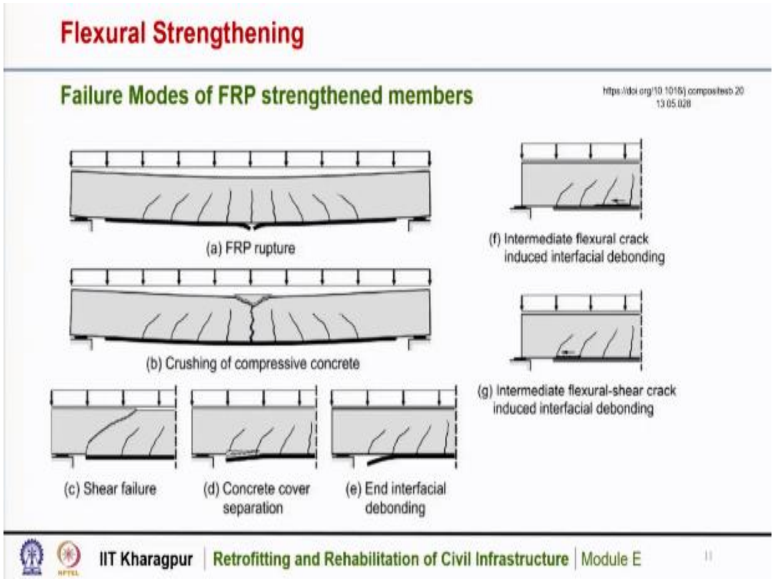

And these are the possible failure modes of the FRP strengthen members, this is FRP rupture, this is a beam we can see it is retrofitted with FRP strips and under the loading there may be rupture of the FRP and with that the member failed. So, this is FRP rupture and this rupture occurs when the strain in the FRP reaches its ultimate strain value.

So, in that condition there is rupture of the FRP and the retrofitted beam fails. There may be crushing of concrete at the compression zone. We can see here that at this location there may be crushing of concrete and with that the beam may failed. So, because of this crushing of concrete the beam fails. These are the typical shear failure, the formation of shear cracks near the end and because of this shear failure the number of shear cracks may appear and with a greater number of shear cracks there may be concrete cover separation.

So, concrete cover separation occurs and that causes failure of the member. This is also due to the formation of number of shear cracks and that causes end interfacial debonding. So, here also near the end debonding may occur due to the greater number of cracks at these locations. These are intermediate flexural cracks appeared on this member and because of the number of flexural cracks there may be debonding at the interface.

At the interface of the FRP and the concrete there may be interfacial debonding with a greater number of cracks. This is also debonding, interfacial debonding due to the presence of a number of flexural shear cracks. So, these are typical flexure shear cracks and with a greater number of this type of flexure shear cracks there may be interfacial debonding of the FRP from the concrete member.

So, these are the possible failure modes, it could be due to FRP rupture or it could be due to crushing of concrete near the compression zone, it could be due to debonding and that debonding may cause concrete cover separation or interfacial debonding or delamination due to the formation of flexure cracks or shear cracks or flexure shear cracks.

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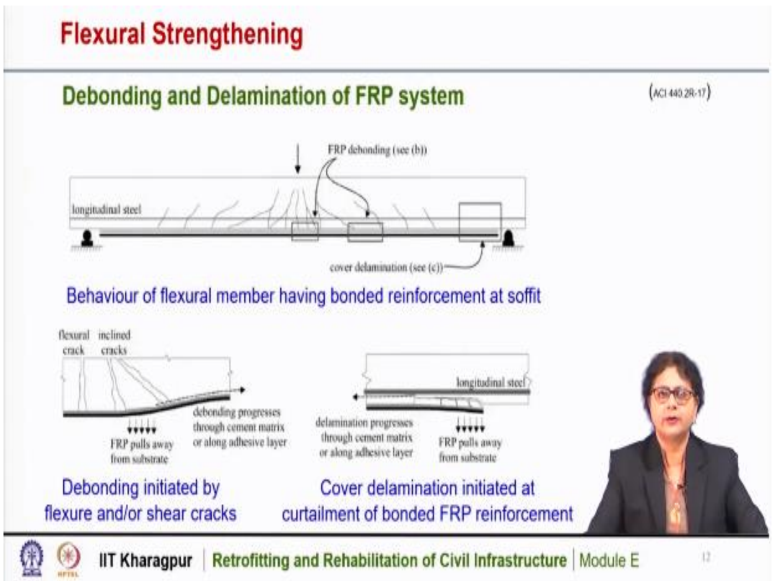

These are the schematic diagram of debonding and delamination of the FRP system. We can see here that this is an FRP retrofitted flexure member and under loading there are a number of flexure cracks and with more number of flexure cracks there may be debonding at this location. So, debonding may occur and that may cause detachment of the FRP from the concrete surface.

Here also covered delamination may be there near the support, so that also due to the formation of cracks near the support. And with more number of cracks debonding can occur and that debonding is initiated by the flexure or shear cracks and we can see here in this schematic diagram that when there is these flexural cracks or flexured shear cracks that FRP will be pulled out from the substrate.

So, there is a pulling of the FRP from the substrate and that causes debonding from the concrete substrate. So, debonding progresses through the cement matrix or along the adhesive layer, so there is debonding at these locations. And this is also debonding with cover delamination and that is initiated at the curtailment of bonded FRP reinforcement.

So, here also there is interfacial debonding, the FRP pulls away from the substrate and the cover concrete is also separated. So, delamination progresses through the cement matrix and along the adhesive layer. So, this is debonding or delamination that may take place with the formation of flexure cracks or shear cracks or flexure shear cracks on the member.

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Now, for the design of flexural strengthening there are several assumptions. The assumptions are design calculation should be based on dimensions, internal reinforcing steel arrangement and material properties of the existing member to be strengthened. Plane section before loading remains plane after loading. There is no relative slip between external FRP and the concrete.

The shear deformation within the adhesive layer is neglected because the adhesive is very thin, so we neglect the shear deformation within the adhesive layer. The tensile strength of concrete is neglected. The maximum compressive strain of concrete is considered as 0.003 and FRP reinforcement has linear elastic stress strain relationship.

So, these are the assumptions for the design of flexure members with FRP retrofitting, that means we are considering that the design calculations should be based on the dimensions, existing steel reinforcement and the material properties of the existing member that we should consider. The plane section before loading remains plane after loading and there is no slip between the FRP and the concrete.

The shear deformation within the adhesive layer is not considered. The tensile strength of concrete is not considered and the maximum compressive strain of concrete is considered as 0.003 and the FRP reinforcement has linear elastic stress strain relationship till failure.

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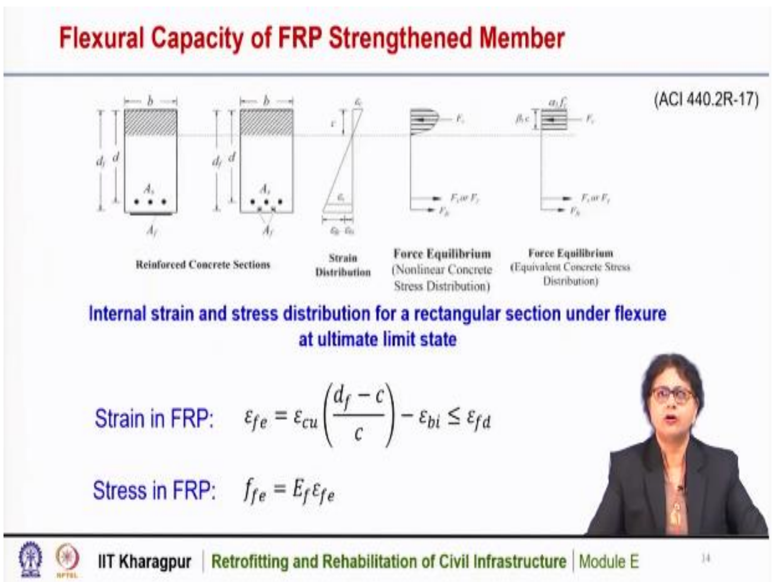

Now, for determining the flexural capacity of FRP strengthen members, we need to consider the internal strain and stress distribution for a rectangular section under flexure at ultimate limit state. So, these are the diagram internal stress strain distribution for rectangular sections under flexure.

So, this is the typical cross section of a reinforced concrete beam and this is the existing steel reinforcement, the area of which is A_s, the width and depth of the beam is b and d_f and A_f is the area of the FRP reinforcement that is provided as external reinforcement. So, this is the FRP strip that is to be provided and this is the area is denoted as A_f.

Similarly, the same principle we can use for NSM, so the area of the NSM reinforcement is A_f and the similar procedure we can use for determining the flexural capacity of FRP strengthened members. This is the strain distribution shown here, this is the strain distribution ε_c is the strain in the concrete, ε_s is the strain in the steel and ε_{fe} is the strain in the FRP and ε_{bi} is the strain in the concrete during the application of the FRP.

So, these are the strain, maximum strain in the members and this is the force equilibrium diagram. So, this is the total compressive force and that is denoted by f_c, the total tensile force is taken by the steel and the FRP and that is denoted by fs and fourth quarter, respectively.

And here it shows that the stress distribution in concrete is non-linear and this is the equivalent concrete stress distribution with rectangular blocks. So, these are the force equilibrium diagram for the rectangular section under flexure. So, the strain in the FRP can be written as $\varepsilon_{fe} = [\varepsilon_{cu}(d_f - c)/c] - \varepsilon_{bi} \leq \varepsilon_{fd}$.

Now, to determine the flexural capacity of the FRP strengthen members, we need to consider the force equilibrium and the strain compatibility. And to determine the flexural capacity we need to find out the depth of the neutral axis and this can be done by a trial-and-error process because the total compressive force should be equal to the total tensile force. And to find out the depth of the neutral axis, we first need to assume a depth and then we can apply the force equilibrium and strain compatibility.

So, from the strain distribution diagram we can write down the strain in the FRP. So, this is the strain in the FRP and this is the strain in the concrete substrate during the time of the FRP application and this is the ε_{cu}.

So, from this similar triangle we can write down the strain in the FRP similarly, the stress in the FRP then can be written as $f_{fe} = E_f \varepsilon_{fe}$, that is the elastic modulus of the FRP into the strain of FRP.

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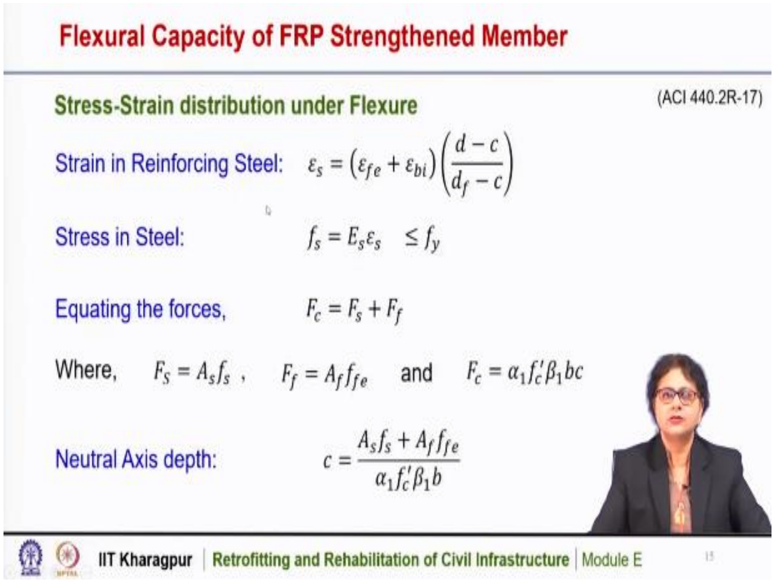

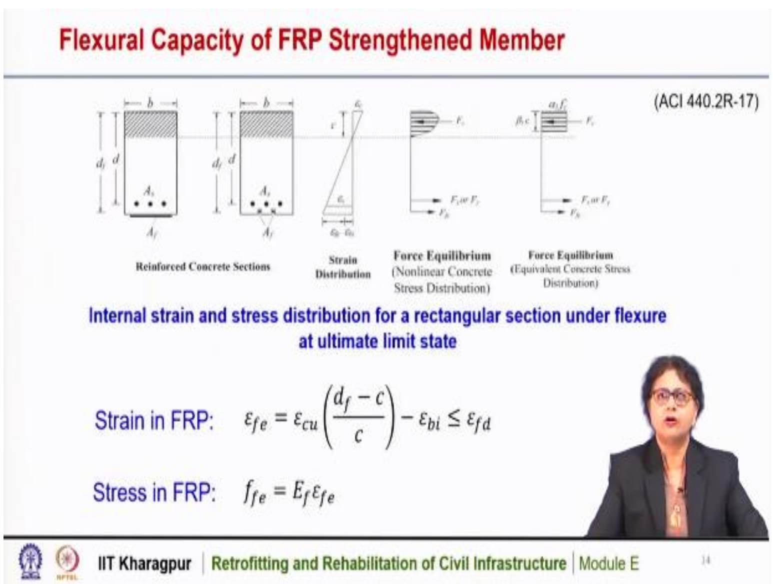

Now, the strain in this reinforcing steel can be similarly written. So, this is the strain in the steel so this can be written similarly as $\varepsilon_s = (\varepsilon_{fe}+\varepsilon_{bi})\ [(d-c)/(d_f-c)]$ and the stress in steel can be written as $f_s = E_s\varepsilon_s \leq f_y$, that is the elastic modulus of steel multiplied with its strain.

Now, we need to equate the forces that is, this is the total compressive force that is taken by the concrete and this is the total tensile force taken by the steel and the FRP. So, we have to equate it so $F_c = F_s + F_f$, where F_s can be written as $A_s \times f_s$ that is the area of the steel reinforcement into the stress in steel. Similarly, the force taken by the FRP is equal to area of the FRP into the stress at the FRP and F_c that is the compressive force is equal to, from this rectangular block we can write it $F_c = \alpha_1 f_c{}' \beta_1 bc$.

So, by putting these values here in this force equation, we can find out the neutral axis depth as $c = [A_s f_s + A_f f_{fe}]/(\alpha_1 f_c{}' \beta_1 b)$. So, this is the neutral axis depth and this can be obtained by trial and error. So, first we have to assume a neutral axis depth and we will equate the total forces and if that is not balancing then we have to go for another trial. So, this way we can find out the depth of the neutral axis and from that the strength of the FRP retrofitted member can be found out.

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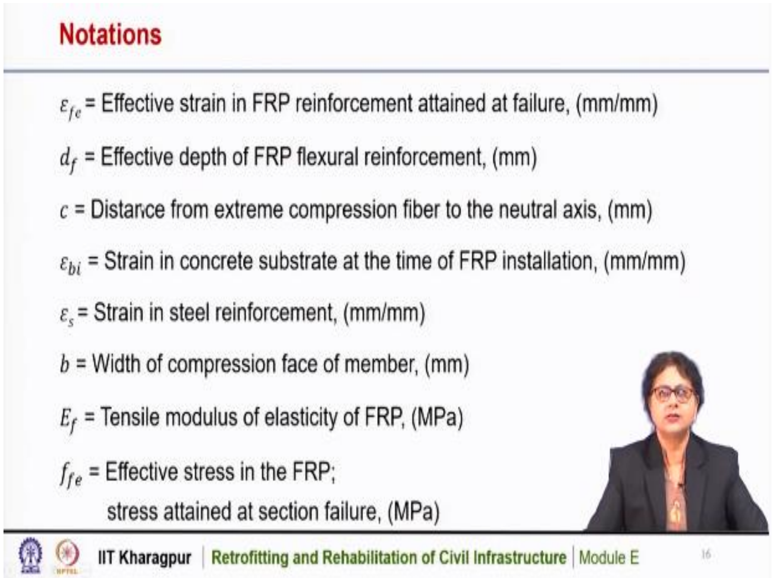

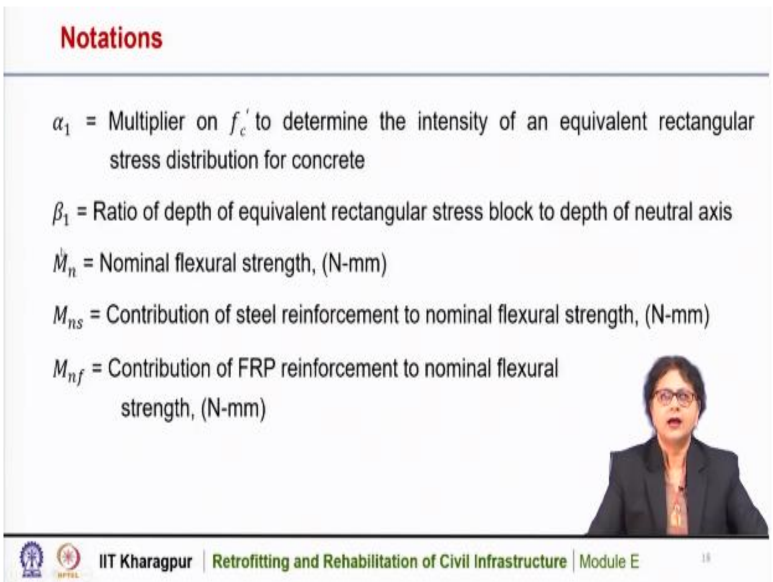

So, these are the notations ε_{fe} is the effective strain in FRP reinforcement attained at failure and these things we have just discussed ε_{bi} is the strain in concrete substrate at the time of FRP installation. And b is the width of the compression phase of the member, because for finding out the force in the concrete we use b.

So, this is width of the compression phase of the member, E_f is the tensile modulus of elasticity of FRP, d is the distance from extreme compression fiber to centroid of steel reinforcement and all these things have been explained, f_y is the specified yield strength of steel reinforcement.

And α_1 is the multiplier on f_c' dash to determine the intensity of an equivalent rectangular stress distribution for concrete and β_1 is the ratio of depth of equivalent rectangular stress block to depth of neutral axis. We will also use these notations M_n is the nominal flexural strength, M_{ns} is the contribution of the steel reinforcement to nominal flexural strength and M_{nf} is the contribution of the FRP reinforcement to nominal flexural strength.

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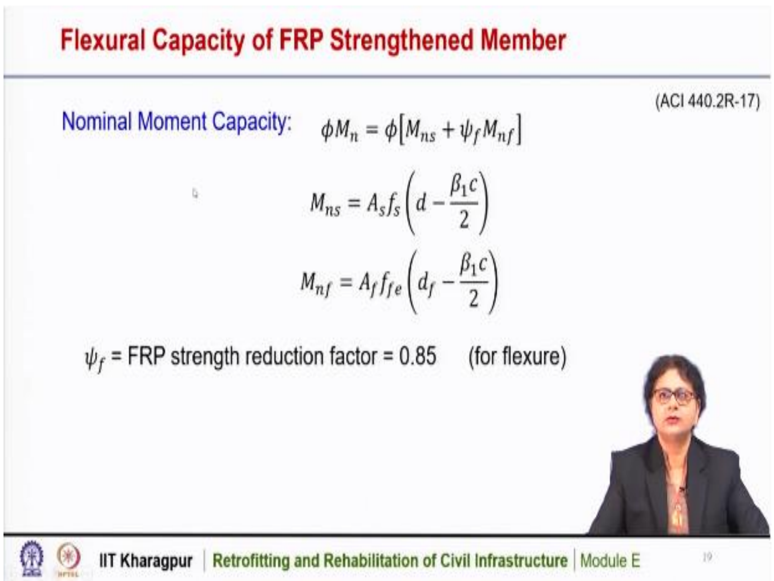

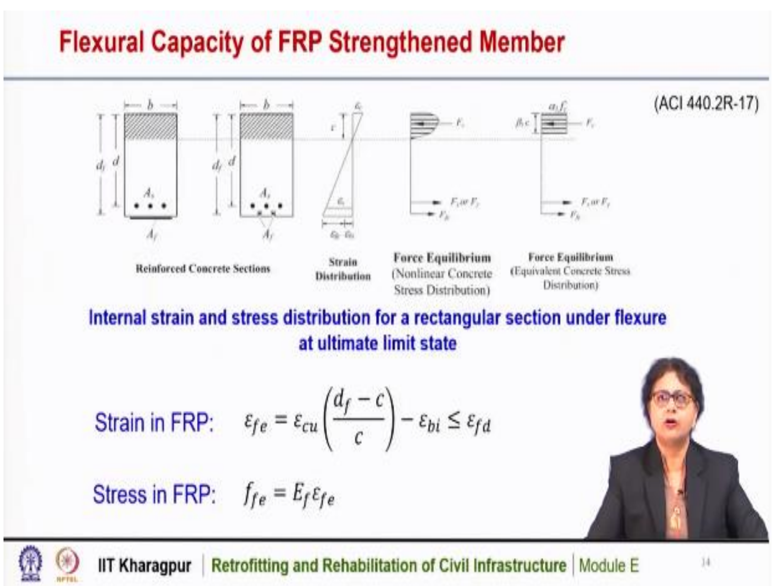

So, the nominal moment capacity can be written as $\phi M_n = \phi\ [M_{ns} + \psi_f M_{nf}]$, where M_{ns} is the strength and that is the contribution of the steel reinforcement and M_{nf} is from the

FRP. So, $M_{ns} = A_s f_s \{d - (\beta_1 c/2)\}$, this is the moment and $M_{nf} = A_f f_{fe} \{d_f - (\beta_1 c/2)\}$. So, from these diagrams we can find out the moment.

ψ_f is FRP strength reduction factor, we have used it earlier and this factor is considered to take care of the uncertainties in the FRP properties and there may be some unevenness in the material or there may be some non-uniformity in the property and that is why another strength reduction factor for FRP has been used. And for flexure this ψ_f has been considered as 0.85.

So, using these factors we can find out the nominal moment capacity of the FRP strengthened member. We have obtained the value of c, that is the depth of the neutral axis and we know the area of steel of the existing member, the area of FRP which is to be provided and from that we can find out the contribution of the steel reinforcement and also the contribution of the FRP reinforcement to the FRP strengthen member. So, from that we can find out the nominal moment capacity as given by this equation.

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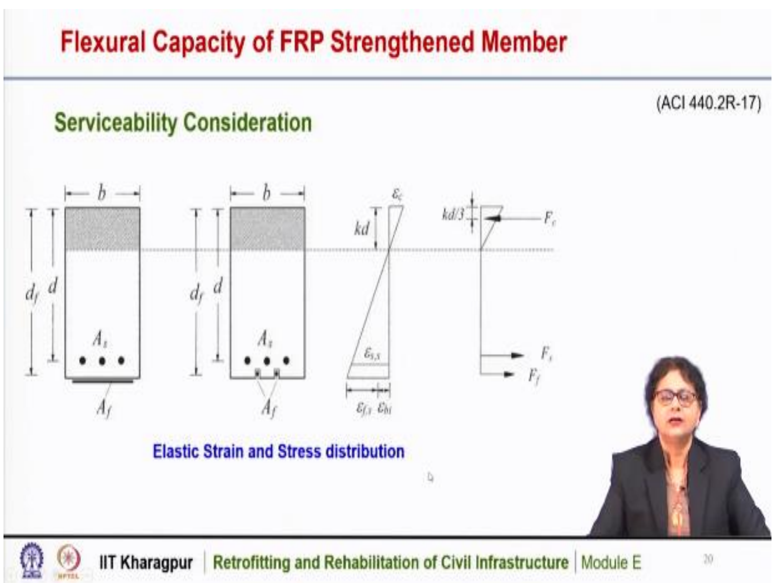

Now, we will also determine the serviceability criteria, the serviceability consideration has been done considering the elastic strain and stress distribution of the rectangular section. So, this is the same rectangular section and for serviceability we will consider that this is elastic strain and stress distribution.

So, this is the compressive strain and this is the strain in the FRP and also in the concrete substrate and this is the strain in the steel and these are the force, compressive force and these are the tensile forces taken by the steel reinforcement and the FRP.

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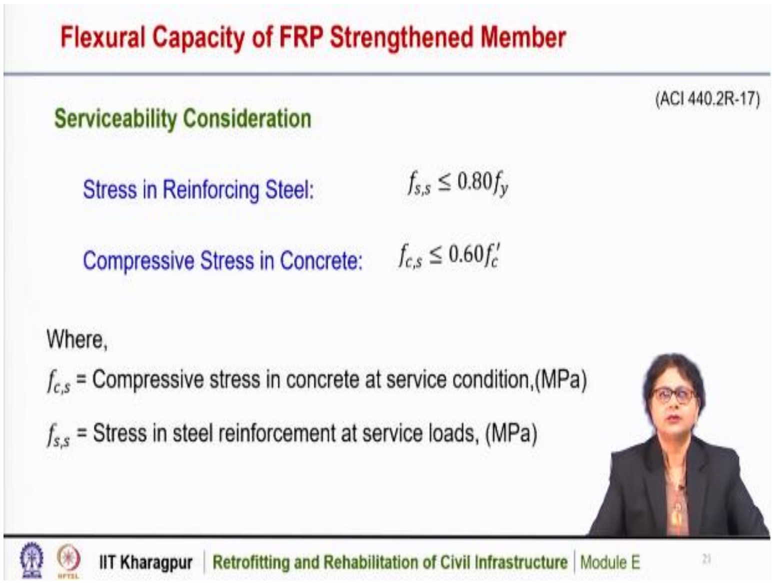

So, the serviceability considerations the stress in the reinforcing steel is denoted as $f_{s,s}$ and $f_{s,s} \leq 0.80 f_y$ and the compressive stress in concrete similarly is denoted as $f_{c,s}$ and $f_{c,s} \leq 0.60 f_c'$. That means for the serviceability consideration the stress in the reinforcement steel should be less than 80 percent of the yield stress of steel and for the compressive stress in concrete it should be less than 60 percent of the compressive strength, $f_{c,s}$ is the compressive stress in concrete at service condition and $f_{s,s}$ is the stress in steel at service loads.

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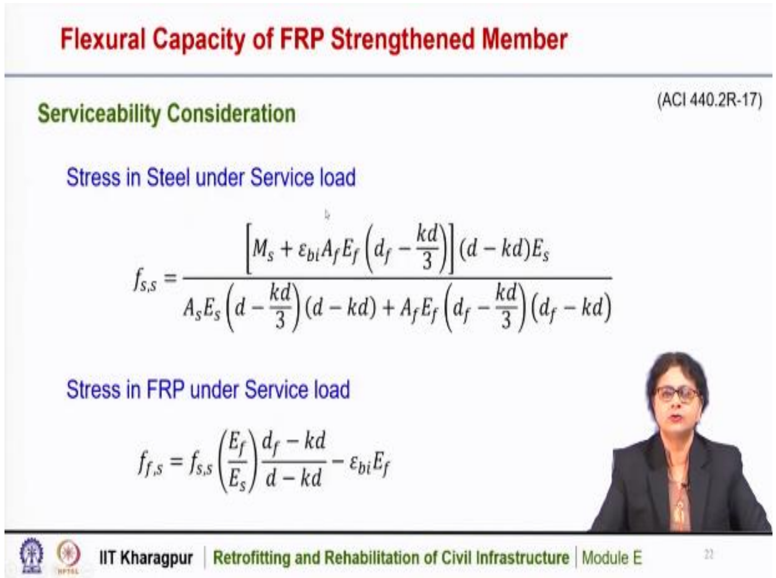

So, from this elastic stress strain distribution diagram we can find out the stress in the steel under the service load and from that diagram we can obtain that $f_{s,s}$ is equal to this so using that stress and strain distribution we can find out the stress in steel under service load. Similarly, the stress in FRP under service load, which is denoted as $f_{f,s}$. So, under serviceability condition the stresses are found in steel as well as in FPR.

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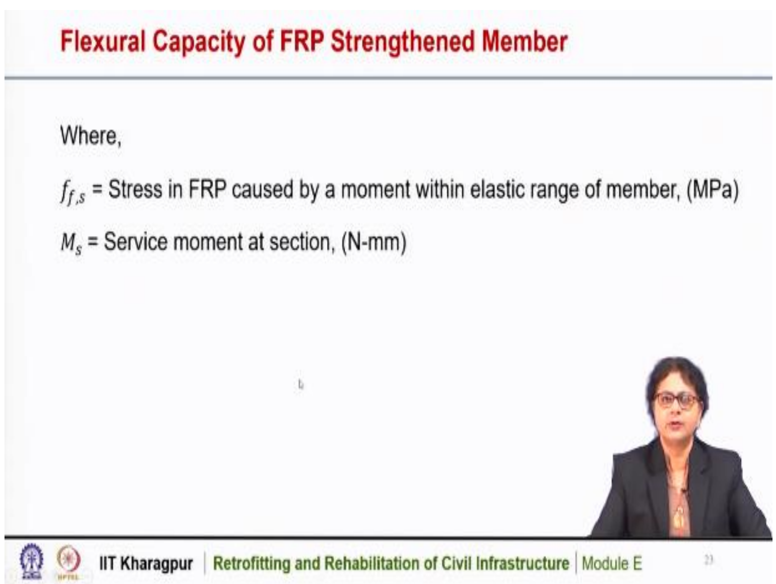

Where $f_{f,s}$ is the stress in FRP caused by a moment within elastic range of member and M_s is the service moment at section and all are in SI units. So, this way we can find out the stresses for the serviceability conditions.

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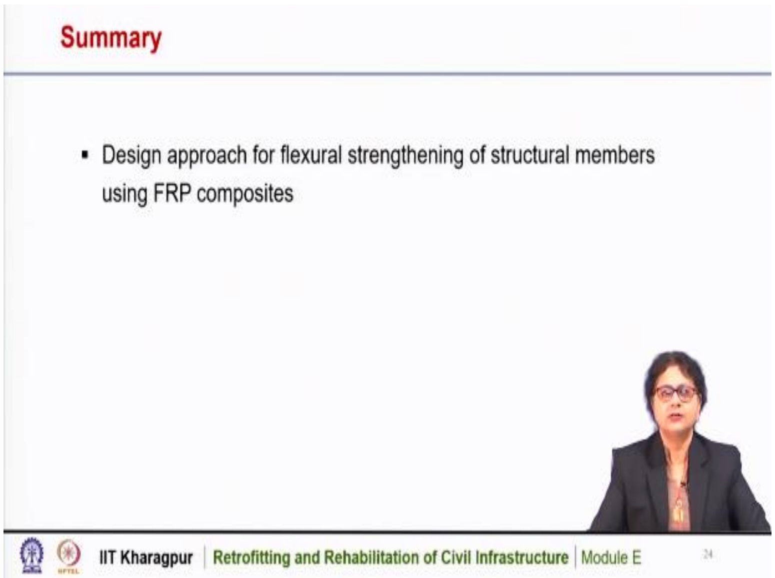

So, to summarize we have discussed the design approach for flexural strengthening of structural members using FRP composites, we have considered the force equilibrium, the strain compatibility and also the different failure modes of the FRP retrofitted flexural members.

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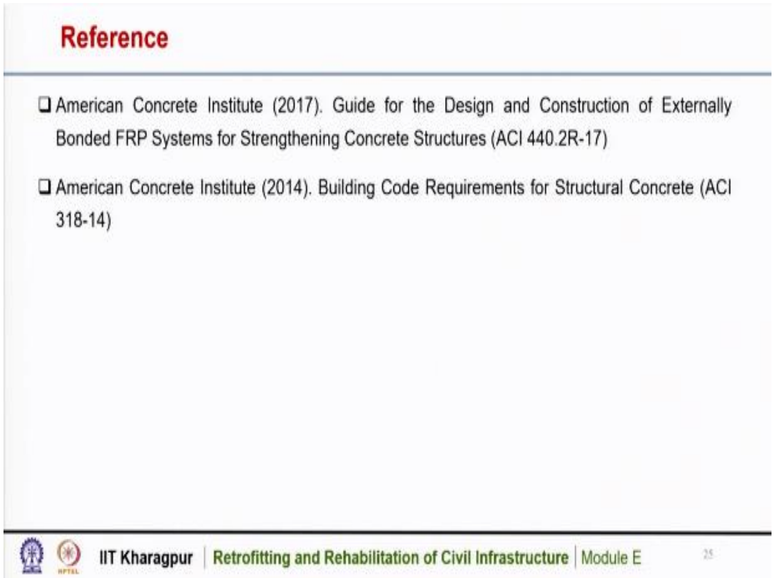

These are the references for today's lecture. Thank you.