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This self-guided presentation covers the use of externally bonded FRP systems for strengthening existing concrete structures. The content of the presentation follows the guidelines given in the ACI 440.2R-08 document.
The presentation will now focus on the engineering principles involved in designing the layout of an FRP strengthening system.
First this presentation will cover flexural strengthening.
The design procedures for flexural strengthening are covered in Chapter 10 of the ACI 440.2R document. Some lab tests have shown increases in flexural capacity from FRP systems of up to 160%. However, when considering serviceability limits, safety factors, and practical issues, it is more reasonable that increases up to 40% can be attained (note that the increases referred to are increases in ultimate or design moment capacity). It is possible to increase both positive and negative moment capacity and both reinforced and prestressed (or post-tensioned) concrete members can be strengthened for flexure. Furthermore, it is generally recognized that FRP reinforcement will improve flexural crack distribution and reduce crack widths, although specific design guidelines for determining this reduction are not currently available in the ACI 440.2R document. The document also does not give specific guidelines on strengthening for flexural loads due to seismic forces.

The objective for any flexural strengthening application is to provide a design moment capacity greater than the moment demand. This is expressed as Eqn (10-1) and is similar to the general requirements given in ACI 318.
The general load-deflection behavior of FRP strengthened flexural members is shown here. Note that increases in FRP reinforcement do result in additional flexural strength being attained. However, increases in FRP reinforcement also result in reduced deformation capacity and ductility. It is also important to note here that increases in FRP reinforcement do not necessarily result in proportional increases in strength.
In order to compute the flexural strength of a member strengthened with FRP reinforcement, the basic principles of reinforced concrete will be employed. As such, the assumptions shown are made in developing the equations for ACI 440.2R. (Note that many of these assumptions are similar to the assumptions used to develop ACI 318.)
For purposes of illustrating the calculations required for determining the strengthening effect of FRP flexural reinforcement, consider the regularly reinforced/FRP strengthened concrete beam shown. The ultimate strength of the beam will be determined based on simultaneously satisfying strain compatibility and internal force equilibrium. The flexural strength of the section will be gained from the contribution of a compressive resultant force in the concrete, the tensile force from the existing steel reinforcement, and the tensile force contribution from the FRP system. This again is very similar to regular steel reinforced concrete design.

\[
M_n = A_s f_s \left( d - \frac{\beta_1 c}{2} \right) + A_f f_{fe} \left( h - \frac{\beta_1 c}{2} \right)
\]
One of the primary design differences between regular steel reinforced concrete and FRP strengthened concrete, is the number and type of failure modes that can occur. All of the failure modes listed must be considered. It is important to note that both failure modes 1 and 4 are brittle, sudden failure modes. It is most often advisable to avoid these failure modes. Failure mode 4, cover delamination, is a unique failure mode and will be dealt with in more depth in the design detailing portion of this presentation.
Like regular reinforced concrete, the flexural behavior of FRP strengthened concrete can be elastic until yielding of the existing steel reinforcement followed by failure initiated by crushing of the concrete in compression. Note that with this failure mode there is still significant deformation (and therefore warning of failure). This is due to the existing steel reinforcement undergoing significant deformation after yielding.
The flexural behavior of FRP strengthened concrete can also be steel yielding followed by failure of the FRP. This can either be failure due to rupture of the FRP (the FRP reaching its ultimate tensile strength) or by FRP debonding off of the surface of the concrete. Again significant deformation is attained by significant post-yield elongation of the existing steel reinforcement. (Also note with this failure mode that once the FRP fails, the beam does still have some residual strength and deflection capacity based on the original unstrengthened section.)
ACI 440.2R gives equations to determine the point at which the FRP material will debond from the concrete. These equations, shown, yield a debonding strain value. This strain is a function of the concrete strength and stiffness of the FRP reinforcement – higher stiffness materials debonding at lower levels of stress than lower stiffness materials. (Also note that the variable “n” is the number of layers of FRP reinforcement, and “t_f” is the thickness of the FRP reinforcement per layer.)
It is important to recognize that the full ultimate strength of the FRP will rarely be realized. If FRP debonding controls failure, only the percentage of ultimate strength will be attained. If concrete crushing controls failure, the concrete will reach its maximum compressive strain before the FRP reaches its rupture strain. Thus, the concept of “effective strain” is introduced in the ACI 440.2R guideline. The “effective strain” is the strain level achieved in the FRP when the section fails (due to concrete crushing, FRP debonding, etc.). Note that since FRP materials are 100% linearly elastic, the effective strain is linearly proportional to the stress developed in the FRP material as well.
For this reason, the moment capacity cannot be calculated by simply “plugging in” the yield strength of the steel, $f_y$, as the stress in the steel and the ultimate strength of the FRP, $f_{fu}$, as the stress in the FRP. The stress, particularly in the FRP, must be determined through an iterative process of simultaneously satisfying strain compatibility and force equilibrium.
The procedure for computing the stresses and ultimately arriving at the moment capacity is performed by the steps shown.
The first step involves calculating the strain in the substrate at the time that the FRP is installed. It is important to realize that the FRP is usually bonded to surfaces that are already stressed. For example when bonding FRP to the bottom of a beam, the bottom of the beam may already be under tension due to its self weight and dead loads. Since the FRP is installed unstressed, it is not capable of resisting these loads that are already in place. For calculation purposes, the state of strain on the substrate when the FRP is being installed should be calculated so that it can be subtracted from the strain in the FRP at increasing levels of load. The initial substrate strain can be computed from the equation shown where $M_{ip}$ is the bending moment in the section due to the existing loads on the member.

\[
\varepsilon_{bi} = \frac{M_{ip} (h - kd)}{I_c E_c}
\]
The second step (and the first step in the iterative calculation procedure), is to estimate the neutral axis depth at ultimate, c. The estimated neutral axis depth will be checked to see if it satisfies both strain compatibility and internal force equilibrium. If these two conditions are not satisfied, the neutral axis depth will need to be revised and checked again. It will first be assumed that strain compatibility is satisfied. Force equilibrium may then be checked.

\[ c \approx 0.20d \]
Given the neutral axis depth and assuming strain compatibility is satisfied, the strain in the FRP can be determined by Eqn (10-3). This equation will also indicate which mode of failure will govern.
With the strain in the FRP determined, the strain level in the steel reinforcement and concrete can be determined.
With the strain level in each material, the stresses in each material can be determined as well. For the steel reinforcement (which is idealized as elastic-perfectly plastic), Eqn 10-9 will indicate the stress level in the steel. For the FRP (which is idealized as perfectly elastic), the stress level can be determined from Eqn 10-4.
It should be recognized, that ACI 318 uses a stress block model for estimating the compressive stress distribution in the concrete – the Whitney stress block. This model is only valid when concrete crushing is governing failure. If FRP failure governs failure, the strain level in the concrete may be substantially lower than 0.003-in/in. The Whitney stress block will not give an equivalent stress distribution for this condition. The actual non-linear stress distribution in the concrete must be considered or an alternative equivalent stress block model must be employed.
One equivalent stress block model, for concrete strains less than 0.003-in/in, is shown here.

\[ \varepsilon_c < 0.003 \]

\[ \beta_1 = 2 - \frac{\frac{4}{(\varepsilon_c/\varepsilon_c') - \tan^{-1}(\varepsilon_c/\varepsilon_c')}}{(\varepsilon_c/\varepsilon_c')\ln(1 + \frac{\varepsilon_c^2}{\varepsilon_c'^2})} \]

\[ \alpha_1 = \frac{0.90\ln\left(1 + \frac{\varepsilon_c^2}{\varepsilon_c'^2}\right)}{\beta_1 \varepsilon_c / \varepsilon_c'} \]

\[ \varepsilon_c' = \frac{1.71f_c'}{E_c} \]
With all of the material stresses determined, internal force equilibrium may be checked. If the neutral axis depth, $c$, determined by the equation shown is different from the estimated neutral axis depth, then force equilibrium is not satisfied. The neutral axis depth must then be revised and the iterative process repeated until force equilibrium is satisfied.
With strain compatibility and force equilibrium satisfied, the nominal moment strength of the reinforced/strengthened concrete section may be determined.

\[ M_n = A_s f_s \left( d - \frac{\beta_1 c}{2} \right) + A_f f_{fe} \left( h - \frac{\beta_1 c}{2} \right) \]
As mentioned before, it is possible to “over-reinforce” a section with FRP reinforcement and cause a significant loss of flexural ductility in the concrete section. It is, therefore, recommended to follow the procedure given in ACI 318 Appendix B to compensate for a loss of ductility with a higher reserve of strength. If the strain in the steel is above 0.005-in/in at failure, the section is viewed to be adequately ductile and a normal strength reduction factor of 0.90 can be used. If the steel does not yield, the section is viewed to be non-ductile and a strength reduction factor of 0.70 should be used. A linear transition between reduction factors should be used when the strain in the steel is between the strain at yield and 0.005-in/in.
With the strength reduction factor, \( \phi \), computed, the design moment strength may be computed. Here we also introduce a “partial” reduction factor applied only to the FRP contribution to the flexural strength. This partial reduction factor is used in recognition of the fact that FRP reinforcement is not as statistically reliable as internal steel reinforcement. The partial reduction factor for FRP flexural reinforcement is 0.85.

\[
\phi M_n = \phi \left[ A_s f_s \left( d - \frac{\beta_i c}{2} \right) + \psi A_f f_{fe} \left( h - \frac{\beta_i c}{2} \right) \right] 
\]

Reduction factor for FRP contribution: \( \psi = 0.85 \)

\[
\phi M_n > M_u \quad (10-1)
\]
For an FRP strengthened section, it is crucial to check the service level stress in the existing steel reinforcement. It is possible to attain an adequate safety factor against flexural failure of a concrete section, but at the same time for service loads to be high enough to cause yielding of the steel. The service level stress in the steel should, therefore, be computed and limited to 80% of the yield strength of the steel.

\[ f_{s,s} \leq 0.80 f_y \quad (9-6) \]
The equations presented here and many of the equations presented in the ACI 440.2R guideline apply specifically to singly reinforced, rectangular concrete sections. The same principles can however be applied to other reinforced concrete members. The same concepts of strain compatibility, force equilibrium, and strain limitations in the FRP reinforcement can be extended to account for compressive steel reinforcement, concrete flanges, and even prestressed steel.
It also bears mentioning that concrete members must have sufficient shear capacity to carry the additional loads associated with the increased flexural capacity. (For that matter, the overall structure should have sufficient capacity as well – columns, load bearing walls, foundations, etc. should be checked for adequate capacity to carry the imposed loads.) If sufficient shear capacity is not available, methods for shear strengthening using FRP systems are also available. This will be discussed next.
The presentation will now focus on the engineering principles involved in designing the layout of an FRP strengthening system.
Shear strengthening involves applying FRP systems with the primary fibers oriented across potential shear cracks (typically vertically in a concrete beam). These external FRP “stirrups” can increase shear capacity by as much as about 2-kips per inch of depth of the section.
The design procedures for shear strengthening are a great deal simpler than those for flexural strengthening and are covered in Chapter 11 of the ACI 440.2R guideline. FRP shear strengthening can be used to increase the shear capacity of beams, columns, walls, and other structures. It can also be used, in some cases, to actually increase the ductility of a member. This is accomplished by providing enough additional shear capacity to change the behavior of members with a shear-dominated failure to a flexural-dominated failure. The flexural-dominated failure results in more ductile behavior.
This diagram better illustrates the ability of FRP shear reinforcement to improve ductility. With low levels of additional FRP shear reinforcement, the shear capacity is increased. With increased levels of reinforcement, the shear capacity exceeds the flexural capacity associated with the applied loads. Thus the steel flexural reinforcement is allowed to yield and the beam exhibits a much more ductile behavior.
FRP shear reinforcement can be in the form of discrete strips or bands of FRP at a certain spacing, or as one large continuous band of reinforcement (space between strips = 0). It can also be oriented with the primary fibers in the vertical direction (perpendicular to the members axis of bending) or inclined in a direction that is closer to being perpendicular to potential shear cracks.
Additionally, the FRP material can be wrapped around the entire cross section, in a “U” wrap configuration, or simply bonded to two sides of the member. (Fully wrapped sections are often impractical for beams as it is necessary to penetrate through the adjoining slab or flange. However, this is quite common and practical for shear strengthening of concrete columns.)
Because of the limited bond area for FRP “U” wraps and FRP bonded only to two sides of the beam, the most common mode of failure is debonding of the FRP. This typically happens at a relatively low level of strain in the FRP, but prior to debonding the FRP can add substantial shear strength to the beam. For almost all fully wrapped applications, the primary mode of failure is loss of aggregate interlock. Most FRP materials have very high elongation capacity, in order to develop these high levels of strain, shear cracks in the concrete would need to become very large. As the crack width increases, eventually a loss in aggregate interlock occurs. With this loss in aggregate interlock, the shear integrity of the concrete diminishes greatly and the shear capacity of the concrete is reduced to nearly zero. This results in an abrupt failure of the member. It is also possible to rupture FRP shear reinforcement, however this is rare. It typically occurs due to high stress concentrations near crack locations or with some very high modulus FRP materials that have relatively low elongation capacity.
This photo shows an test specimen strengthened with FRP shear reinforcement in the form of “U” wraps. Note the failure that is exhibited by debonding of the FRP down to the location of the shear crack.
With FRP shear reinforcement, the concept of strain limitations or effective strain is again employed. For fully wrapped applications, loss of aggregate typically controls the failure. This is assumed to occur at a strain level of 0.004-in/in. Thus the effective strain in these applications is assumed to be 0.004-in/in. (Note that the improbably FRP rupture failure mode is accounted for by limiting the effective strain to 75% of the ultimate elongation of the FRP as well.) As mentioned before, some bond u wraps or face plies applications will fail by loss of aggregate interlock as well (effective strain = 0.004-in/in). However, they will most often be controlled by debonding. Here again the strain at which the FRP will debond is represented by a percentage, kv, of the ultimate strain in the FRP.
The percentage, $k_v$, is a function of the strength of the concrete and wrapping scheme used. This is evident in the series of equations shown. Note also that the effective strain will be limited again to 75% of the ultimate elongation to account for the less common FRP rupture failure mode.

The effective strain is also a function of the active bond length, $L_e$. This length is a function of the stiffness of the FRP material and is described in the next slide…
It has been observed that when FRP bonded to concrete is in direct tension (without curvature that would be present in FRP flexural reinforcement), the majority of bond stresses are carried over a relatively small length of the FRP. Once the bond capacity of the FRP to the concrete in this region is exceeded, the bond length shifts backward along the bonded length of the FRP, as this bond capacity is subsequently exceeded, the bond length shifts again. This “unzipping” continues until the entire length of the FRP strip has debonded. The critical concept here is that the force required to debond the material over the active bond length will result in the entire length of the FRP strip debonding.
Translating this to shear reinforcement, the majority of the bond stresses will be carried by the regions shown in this diagram.
With the effective strain computed, the shear contribution of the FRP reinforcement can be simply calculated by the equations shown. Note that these equations are very similar to the equations used to compute the contribution of internal steel stirrups to the shear strength.

\[
V_f = \frac{A_{fv} f_{fe} (\sin \alpha + \cos \alpha) d_{fv}}{s_f} \quad (11-3)
\]

\[
A_{fv} = 2nt_f w_f \quad (11-4)
\]

\[
f_{fe} = \varepsilon_{fe} E_f \quad (11-5)
\]
The diagram shown shows graphically the stain limitation imposed on FRP shear reinforcement. Note that the strain limitation is constant for fully wrapped applications, whereas for bonded applications the strain limitation becomes more aggressive with increasing stiffness (or thickness) of the FRP material.
Translating this to the shear contribution from the FRP, for fully wrapped applications, increasing the thickness (or number of layers) of FRP results in a proportional increase in shear strength added. For bonded applications, increasing the thickness of the FRP, results in ever lesser increases in shear strength. Thus, in general, fully wrapped applications will be more effective at increasing shear capacity than bonded applications.
The overall design shear capacity can be obtained from summing the contribution of the concrete, steel, and FRP as shown in Eqn 11-2 with the appropriate strength reduction factors. Note that the overall strength reduction factor, phi, is the same as for regular steel reinforced concrete and that a partial reduction factor is again applied to the FRP. The partial reduction factor on the FRP is 0.85 for bond-critical applications and 0.95 for contact-critical applications. This is done in recognition of the fact that fully wrapped applications are more statistically reliable than bonded applications.

\[
\phi V_n = \phi \left( V_c + V_s + \psi V_f \right) \quad (11-2)
\]

\[
\phi = 0.85 \quad (ACI 318)
\]

\[
\psi_f = 0.95 \quad (\text{fully wrapped})
\]

\[
\psi_f = 0.85 \quad (\text{bonded U - wraps or face plies})
\]
As a final note on shear strengthening, it is important to maintain reasonable spacing of FRP stirrups just as with steel stirrups. The maximum spacing that will ensure that the FRP reinforcement crosses a potential shear crack is given in the equation shown. There is also an upper limit on the total amount of shear reinforcement that can be provided given by Eqn (11-11). This criteria is similar to the criteria given by ACI 318 to prevent crushing of the compression strut developed in the concrete under shear loads.
The presentation will now focus on the engineering principles involved in designing the layout of an FRP strengthening system.
The final application for FRP strengthening systems covered by the ACI 440.2R document is confinement of members subjected to axial compression. FRP reinforcement in this application mimics the effects of internal steel ties or spiral reinforcement. These FRP “jackets” can result in modest increases in the axial load bearing capacity of some columns and can dramatically improve the ductility of columns.
The engineering procedures for designing FRP confinement are given in Chapter 12 of the ACI 440.2R guideline.
The effect of providing additional confinement using FRP strengthening systems is to actually alter the stress strain behavior of the concrete itself. With normal, unconfined concrete, the concrete reaches a certain compressive strength, then as the concrete cracks there is a modest amount of strain softening until the concrete finally fails in compression. By confining the concrete with FRP, the dilation of the concrete is restrained. This restraint of dilation exerts a confining pressure on the concrete and allows it to take additional load and undergo additional deformation. The result is both increases in the apparent strength of the concrete and increased ductility. Note again that this is the result of restraint provided by fibers principally oriented transverse to the longitudinal axis of the column. Any contribution of longitudinally aligned fibers to the axial compression strength of a concrete member should be neglected.
This diagram represents the behavior of unconfined vs FRP confined concrete. The FRP confinement allows the concrete to take additional stress and to undergo additional longitudinal strain. Thus, FRP confinement has the effect of strengthening the member in compression and increasing its displacement ductility. Note that we will again incorporate an effective strain level in the FRP which is smaller than its ultimate strain capacity.

In order to quantify the increases in strength and ductility, we will first develop a model for the behavior of FRP confined concrete.
The effective strain in the FRP for members subjected to pure axial loads is limited based on the variation in the FRP strain versus the transverse strain in the concrete. It is recognized here that there may be stress concentrations at cracks, offsets, or edges that may result in FRP failure before a transverse strain corresponding to the rupture strain of the FRP is reached in the concrete.

Additionally, for members subjected to axial forces plus bending moments, the transverse strain in the concrete needs to be limited to prevent loss of aggregate interlock. Similar to the concept presented for shear strengthening, the effective strain is limited to 0.004-in/in to maintain shear integrity of the concrete.
The generalized model for FRP confined concrete is defined here. The model essentially describes bi-linear stress strain behavior. Note that the behavior of FRP confined concrete matches that of unconfined concrete in the elastic region. Once the unconfined concrete cracks and begins to exhibit strain softening, the FRP jacket is engaged and allows the FRP confined concrete to instead continue to resist additional compressive forces and continue to deform.

\[ f_c = \begin{cases} E_c \varepsilon_c - \frac{(E_c - E_2)^2}{4f_c'} \varepsilon_c^2 & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_i' \\ f_c' + E_2 \varepsilon_c & \text{for } \varepsilon_i' \leq \varepsilon_c \leq \varepsilon_{cu} \end{cases} \]  

(12-2a)

Where,

\[ E_2 = \frac{f_c' - f_c}{\varepsilon_{cu}} \]  

(12-2b)

\[ \varepsilon_i' = \frac{2f_c'}{E_c - E_2} \]  

(12-2c)
The maximum compressive strength and compressive strain that can be achieved are shown here. These are a function of the confining pressure exerted by the FRP and shape factors that are a function of the cross-sectional shape and dimensions of the element.
For circular sections, the confining pressure can be attained simply by the statics of a thin-walled pressure vessel. The confining pressure at the level of effective strain in the FRP is given by Eq 12-4. FRP confinement is most effective at confining circular sections, and the shape factors can be taken as 1.0.

\[ f_i = \frac{2E_j n t_f \varepsilon_{fe}}{D} \quad \text{(12-4)} \]

**Shape factors:**
\[ \kappa_a = \kappa_b = 1.0 \]
For rectangular sections, the confining pressure is again defined using the statics of a thin walled pressure vessel. Only here an equivalent circular column is used with a diameter equal to the hypotenuse of the two sides of the rectangular section.
The shape factors are then defined based on the aspect ratio \((b/h)\) and the effective confinement area versus the total area of the concrete. The confining stress from the FRP jacket is concentrated at the corners of the jacket as shown. The areas along the length of each side are effectively unconfined.

\[ \kappa_d = \frac{A_e}{A_c} \left( \frac{b}{h} \right)^2 \]  
\[ \kappa_b = \frac{A_e}{A_c} \left( \frac{h}{b} \right)^{0.5} \]
The unconfined area is assumed to be parabolic, and the ratio of the confined area to total area can be computed from Eq 12-11.
It is important to note that a minimum amount of FRP reinforcement needs to be used to ensure the bilinear model described. For low levels of FRP confinement, a strain softening is still evident. In order to ensure that an ascending branch will be attained (and therefore an increase in strength), the ratio of the confining pressure to the unconfined compressive strength should be greater than 0.08.

\[ \frac{f_l}{f'_c} > 0.08 \]
The confinement model can be used to substitute the compressive strength values in ACI 318 equations for axial compression with the confined concrete compressive strength values.
The confinement model can also be used to develop axial-moment interaction behavior. The increase in ultimate compressive strain capacity results in increased rotation and displacement ductility. The increase in compressive strength results in increases in axial load carrying capacity.
When looking at the axial-moment strength interaction, it is important to note that the strengthening effect is only significant for compression controlled members. The effect of FRP confinement should be neglected for tension-controlled sections.
The presentation will now focus on the engineering principles involved in designing the layout of an FRP strengthening system.
Chapter 9 of the ACI 440.2R guideline covers general guidelines on the use of FRP materials.
The reasonable limits to strengthening are covered in this section of the guideline. The limits to additional strength which may be gained may be controlled by the strength of other structural components or by other failure mechanisms. These must always be considered when approaching any strengthening application. Additionally, ACI 440.2R recommends a certain baseline strength from the existing structure to be a viable candidate for FRP strengthening. Per Eqn 9-1, ACI 440.2R recommends that the existing structure be able to sustain 110% of the dead load and 75% of the live load without FRP reinforcement. This is to guard against structural collapse should the FRP material be lost.
Furthermore, it is known that current FRP strengthening systems are susceptible to complete loss in a fire. However, insulation materials can be effective in insulating the existing reinforced concrete structure and thus delaying its degradation in a fire. Also, the contribution if the FRP system can be considered if it is demonstrated that the FRP temperature remains below a critical temperature.
Thus, when a structural fire rating is required, it is recommended to further analyze the structure according to ACI 216 to determine the fire endurance of the structure. ACI 216 involves determining the reduced strength of the reinforcing steel and concrete in a fire, then calculating the associated reduction in strength of the member, and then ensuring that the reduced strength of the member can sustain at least its service loads.
This concept can be extended to include FRP systems. The reduced strength of the concrete and steel can be found using the resources in ACI 216 and the reduced strength of the FRP can be taken as zero. With these material properties, the strength of the member can be computed and compared against the load effects from the live and dead load according to Eqn 9-2.

* Please note that the standard allows for the strength to be included if the temperature of the FRP remains below a critical temperature.
There are two types of applications for FRP that require two different levels of surface preparation. Contact critical applications require only intimate contact between the FRP System and the concrete. For this type of application, you generally need clean, sound, dry concrete.

Bond critical applications require an adhesive bond between the FRP system and the concrete. The surface preparation for this type of application is more involved.
Development Length

- The bond capacity of FRP is developed over a critical length:

\[ l_{df} = 0.057 \sqrt{\frac{nE_f t_f}{f'_c}} \]

in in.-lb units

\[ l_{df} = \sqrt{\frac{nE_f t_f}{f'_c}} \]

in SI units
Reinforcement Details
Chapter 13, Guide

- General Guidelines:
  - Do not turn inside corners;
  - Provide a minimum 1/2 in. (13 mm) radius when the sheet is wrapped around outside corners;
  - Provide adequate development length;
  - Provide sufficient overlap when splicing FRP plies.
Bond and Delamination

- FRP debonding
  - Initiate at flexural and/or flexural-shear crack near maximum moment
  - Under loading these cracks open and induce high interfacial shear stress causing FRP debonding

- FRP end peeling
  - can results from the normal stresses developed at the end
  - cover cover layer splitting from at the level of the tensile reinforcement
FRP debonding can occur in a variety of modes. Debonding can initiate at the location of cracks and then progress along the length of the FRP. It can also initiate at the curtailment of the FRP and debond the cover layer of concrete. With this failure mode, bond stresses normal to the surface of the FRP act to put the concrete section under direct vertical tension. This tensile force can rupture the concrete at its weak plane of tension which is the location of the first layer of steel reinforcement (cover distance). This failure causes the entire cover layer of concrete to delaminate from the rest of the beam.
The normal forces that cause cover tension delamination are the result of high stresses at the termination point of the FRP reinforcement. For this reason it is recommended to stagger the termination of multiple plies of reinforcement (where applicable) and to terminate the reinforcement in low stress regions. For continuous beams, it is best to terminate the reinforcement past the inflection point.
The normal forces that cause cover tension delamination are the result of high stresses at the termination point of the FRP reinforcement. For this reason it is recommended to stagger the termination of multiple plies of reinforcement (where applicable) and to terminate the reinforcement in low stress regions. For continuous beams, it is best to terminate the reinforcement past the inflection point.
For simply supported beams, the reinforcement should be extended past the point representing the cracking moment on the moment diagram. If the termination is in a high shear zone, additional anchorage reinforcement should be provided.
The anchorage reinforcement can be in the form of “U” wraps. The area of anchorage reinforcement can be determined from the equation shown.
Bond and Delamination
Transverse ("clamping") Reinforcement

- Area of transverse ("clamping") FRP U-Wrap reinforcement to prevent concrete cover layer from splitting:

\[ A_{f,\text{anchor}} = \frac{(A_f f_{fu})_{\text{longitudinal}}}{(E_f \kappa_v \varepsilon_{fu})_{\text{anchor}}} \]
This self-guided presentation covers the use of externally bonded FRP systems for strengthening existing concrete structures. The content of the presentation follows the guidelines given in the ACI 440.2R-08 document.