INCREASING BOND STRENGTH BY CONFINING CONCRETE BEAMS WITH CFRP COMPOSITES

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Abstract -
This research studies the behavior of steel Reinforced Concert (RC) beams that are subjected to bond deterioration due to corrosion of steel bars, and to investigate the effectiveness of utilizing Carbon Fiber Reinforcement Polymer (CFRP) in increasing the bond strength in order avoid a bond failure. Corrosion is the primary limit state in steel Reinforced Concrete beams in severe environments. Corrosion of steel bars causes a decrease in steel bar section, loss of bond, and delamination of concrete cover. The bond between the steel reinforcement and the surrounding concrete is extremely important to ensure a stress transfer between concrete and reinforcing bars. The author concludes that confining of concrete beams will increase the bond between the steel rebars and the surrounding concrete.

I. INTRODUCTION

The most commonly used construction material is concrete, as a matter of fact, concrete is used twice as much as all other construction materials combined. Moreover, globally and only second to water, concrete is the most consumed material, averaging about one cubic meter for every person on Earth (Gagg 2014). With mixture of 60 – 65% aggregates like sand, gravel, and crushed stone and 15 – 20% water, cement constitutes only 10 – 15% of the concrete mixture.

Since concrete enjoys large compression strength but low tensile strength, steel reinforcing bars (rebars) are embedded in the concrete, primarily, in regions where the concrete experiences tensile stresses. This is because steel withstands large tensile and compression stresses. The steel bars absorb the tensile stresses while the structural member relies on concrete to resist compression. Another structural advantage of strengthening concrete with steel reinforcement is that while concrete is brittle, steel is ductile. Where ductility is defined as the ability of a structural material to undergo large deformations before rupture. Maintaining ductility is essential for safety measure during structural design to ensure that the structure will demonstrate external evident traces of overloading (Bilek*et al.*2017). When external loads are applied on the concrete surface, a complete bond between the concrete member and the embedded rebars must be secured. This will make sure that the two materials act in a composite manner, ensure a complete transfer of tensile stresses from the concrete to the steel rebars, and guarantee a ductile structural behavior (Verbeck 1975; Rijal*et al.* 2015).

The corrosion process is an electrochemical mechanism in which metal is reduced to a lesser state of energy, that being its natural ore. Various causes exist; the most predominate being chlorides, carbonization, and oxygen.Corrosion of steel in a reinforced concrete structure leads to cracking of the concrete cover, reduction of steel cross-sectional area, loss of concrete integrity, and loss of bond between steel bars and surrounding concrete. This reduces strength and ductility, which shortens the service life of the structure.

II. BOND VS UNBOND

Bond is considered a force transfer mechanism according to the American Concrete Institute (ACI318-19). A perfect bond between the steel reinforcement and the surrounding concrete is essential to ensure a complete transfer of stresses and allow the structural member to reach its design strength. The tensile stress transferfrom concrete to the embedded steel rebars.

The bond mechanisms are defined as friction, chemical adhesion, and mechanical bond:

- Friction is created by the contact between the rebar surface and the surrounding concrete, and it is considered the main bonding mechanism in plain rebars. Friction is developed as the rebars tend to slip within the concrete as the beam deflects under the applied loads. A rough rebar surface texture with slight rust will improve friction.
- Chemical adhesion is developed by the chemical interaction between the steel rebars and the concrete paste, and provides a minor bond strength with relative to other bond mechanisms.
- Mechanical bond is the most effective bond mechanism in deformed bars and is developed when the bar deformations (ribs) create a mechanical interlock with the surrounding concrete.

The bond strength, however, is controlled by the following factors: confinement of concrete; concrete quality and strength ; concrete cover thickness; diameter, shape, spacing, and roughness of steel reinforcement; and mechanical anchorage by means of hooks beyond end supports, splices, and development lengths

The bond deterioration between steel bars and the surrounding concrete is caused by the breakdown of the passive layer on the surface of the steel reinforcement. When a rebar corrodes, its volume initially increases by a factor of 8 to 12 time the original volume of the rebar. As a result, it produces extremely large radial stresses on the concrete. When the internal radial stresses exceed the tensile strength of concrete, cracks form between the corroded rebar and the closest exterior surface. As the corrosion advances, it significantly decreases the size and the yield strength of the rebar. In addition to loss of cross sectional area and mechanical properties, rebar corrosion destroys the bond between the rebar and the surrounding concrete, allowing the steel to slip within the concrete. Which results in significant decrease in the ability of the rebar to transfer forces as the cross-section fails to behave in a composite manner.

2.1 CFRP Sheets

Due to lack of adequate knowledge of durability of concrete structures, many structures constructed in the United States prior to 1970s with little attention to durability issues, e.g. corrosion of rebars in concrete, require immediate repair and retrofit. Several conventional repair materials and systems have been used. However, currently, the most promising repair techniques is the use of Carbon Fiber Reinforcement Polymer (CFRP) composites.

CFRP composite materials are composed of fiber reinforcement bonded together with a resin matrix. The fibers provide the composite with its unique structural properties. The resin matrix supports the fibers, protect them, and transfer the applied load to the fibers through shearing stresses. The mechanical properties of composites vary with the amount and orientation of the fiber in different directions.Most of the commercially available CFRP systems in the construction market consist of uniaxial fibers embedded in a resin matrix, typically epoxy. Multi-directional fiber reinforced polymer composite systems are also available. All fibers used in structural retrofit are very small in diameter. Despite their small size, carbon fibers exhibit very high tensile mechanical properties. High modulus (HM) carbon fiber exhibits the highest stiffness, while the high strength (HS) carbon exhibits the highest strength. Carbon fibers have limited ultimate strain, which may limit the deformability of strengthened members. However, in concrete buildings, local debonding between FRP sheets and concrete substrate would allow for acceptable level of global deformations before failure.Unlike steel, carbon fibers are elastic to failure.

Properties of advanced composite materials which are very useful in structural engineering applications include the followings:high strength/weight ratio; high stiffness/weight ratio; resistance to chemical attack; corrosion resistance; magnetic permeability; good fatigue resistance; controllable thermal expansion characteristics; unique and controllable damping characteristics; and low overall and life-cycle cost. High strength/weight and stiffness/weight ratios are particularly important because less volume of the material is used when compared to other materials. Lighter unit weight implies less addition of dead load to the building. Other advantages include less equipment and manpower and thus reduced installation cost.

III. BOND STRENGTHENING OF CONCRETE BEAMS WITH CFRPS SHEETS

Many factors affect the bond between steel rebars and concrete. Bond force increases with the increase of concrete compressive strength and with confinement of concrete (ACI 408R-03).Confinement of concrete increases the ductility and compressive strength of concreteand is accomplished by wrapping the concrete in the direction perpendicular to the compressive/tensile stresses.Having flat surface, plates or sheets, CFRP systems are ideal for maintenance and retrofit applications on concrete components. FRP systems have very high tensile strength. They could simply be classified as a form of tension reinforcement, e.g. steel reinforcing bars. In addition, they are ideal for confining concrete members.

3.1 Increase of Bond Strength Due to Confinement of Concrete

The primarily three categories that control the bond stress–slip behavior are structural characteristics, bar properties of reinforcing bars, and properties of concrete. Structural characteristics are explained by the geometrical arrangement of the reinforcement which contains, bar spacing, development length, concrete cover, concrete confinement degree, and lap splices. properties of reinforcing bars contain the bar diameter and location, yield stress of the steel, and roughness of the bar surface. Properties of concrete include tensile and compressive strength, aggregate type and size, consolidation degree, workability of concrete, slump, admixtures, and consolidation degree. (ACI 408R-03).Confinement of concrete increases the bond strength, and as a result, it modifies the bond stress–slip relationship as it shifts the splitting mode of failure to a pullout failure. This is because confinement of concrete plays a major rule in reducing the deterioration of post bond failure; which delays and controls crack propagation when compared to unconfined concrete.

3.1.1 Bond Strength

Equations for bond strength have been developed by various researchers based on comparisons with experimental data. Even thoughthat these researchers developed their expressions based on comparison with test results conducted by them (Orangun*et al.* 1977; Kemp and Wilhelm 1979; Darwin *et al.*

1996a,b;Esfahani and Rangan 1998a,b; and Zuo and Darwin 2000), these equations provide reasonably accurate prediction of bond strength (ACI 408R-03). Below are most of the equations developed by various researchers.

Orangun*et al*. (1977)	$\tau_c = \left[1.22 + 3\frac{c_{min}}{d_b} + 50\frac{d_b}{l_d}\right]\sqrt{f_c'}$	Eq. 1
	$\tau_b = \left[1.22 + 3\frac{c_{min}}{d_b} + 50\frac{d_b}{l_d} + \frac{A_{tr}f_{yt}}{500snd_b}\right]\sqrt{f_c'}$	Eq. 2
Kemp and Wilhelm (1979)	$\tau_b = \left[6.57 + 2.9\frac{c}{d_b}\right]\sqrt{f_c'} + 1.1\frac{A_{tr}f_{yt}}{s}$	Eq. 3
Darwin *et al.* (1996a)	$T_c = [63l_d(c_{min} + 0.5d_b) + 2130A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right)\sqrt[4]{f_c'}$	Eq. 4
	$T_b = \left[[63l_d(c_{min} + 0.5d_b) + 2130A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) + 2226t_r t_d\frac{NA_{tr}}{n} + 66\right]\sqrt[4]{f_c'}$	Eq. 5
Zuo and Darwin (1998, 2000)	$T_c = [59.8l_d(c_{min} + 0.5d_b) + 2350A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right)\sqrt[4]{f_c'}$	Eq. 6
	$T_b = \left[[59.8l_d(c_{min} + 0.5d_b) + 2350A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) + (31.14t_r t_d\frac{NA_{tr}}{n} + 4)\sqrt{f_c'}\right]\sqrt[4]{f_c'}$	Eq. 7
ACI Committee 408 (2003)	$T_c = [59.9l_d(c_{min} + 0.5d_b) + 2400A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right)\sqrt[4]{f_c'}$	Eq. 8
	$T_b = \left[[59.9l_d(c_{min} + 0.5d_b) + 2400A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) + (30.88t_r t_d\frac{NA_{tr}}{n} + 3)\sqrt{f_c'}\right]\sqrt[4]{f_c'}$	Eq. 9
FIB Model Code (2010)	$\tau_{c,b} = 54\left(\frac{f_c'}{25}\right)^{0.25}\left(\frac{l_d}{d_b}\right)^{0.55}\left(\frac{25}{d_b}\right)^{0.2}\left[\left(\frac{c_{min}}{d_b}\right)^{0.25}\left(\frac{c_{max}}{c_{min}}\right)^{0.1} + k_m k_{tr}\right]$ (MPa)	Eq. 10

3.1.2 Bond Stress–Slip Model

Since concrete shrinks, unlike steel, it will create small internal cracks near bar surface. This is due to the relative displacement of concrete as it shrinks with relative to the rebar. As these cracks are expected to form before the bar ribs, they will cause stress concentration at small loads. These cracks are likely to spread to the surface at a beam end and possibly separate a small region of concrete from the rest of the beam. Longitudinal cracks propagate with the increase in loading, causing spalling of concrete cover presented as softening in the bond stress–slip curve. These cracks will cause damage to the concrete adjacent to the bar ribs, and a local slip will occur. At higher loads, the concrete before the ribs may crush allowing the bar to limitedly translate within the concrete. High confinement of concrete will cause all of the concrete between ribs to crush and shear cracks to occur along the border of the bar resulting in a pullout failure at higher loads when compared to splitting failure.Pullout failure occurs in specimens with high confinement (ACI 408R-03). Whether confined with steel or FRP composites, bond stress–slipbehaviorof concrete confined with FRP or steel has been investigatedby several researchers (Eligehausen*et al.* 1983; Filippou*et al.*1983; Esfahani and Rangan 1998b; Harajli and Mabsout 2002; Harajli 2004; Harajli 2009; FIB Code, 2010). The most common bond stress–slip constitutive curves are those suggested by Harajli(2009) and FIB Model Code (2010).

Harajli(2009)presented a constitutive model for bond stress–slip relationship of ribbed steel bars in concrete confined in FRP sheets. The proposed model consists of a monotonic curve representing the pull–out mode of failure, and two curves representing the splitting bond failure as shown in **Error! Reference source not found.**.The pull–out curve is presented as follows.

$$\tau = \tau_b \left(\frac{s}{s_1}\right)^{0.3} (\text{MPa}) \text{for} \quad 0 < s < s_1 \qquad \text{Eq. 11}$$

$$\tau = \tau_b (\text{MPa}) \ \text{for} s_1 < s < s_2 \qquad \text{Eq. 12}$$

$$\tau = \tau_b - (\tau_b - \tau_f)\frac{s - s_2}{s_3 - s_2} (\text{MPa}) \quad \text{for} s_2 < s < s_3 \qquad \text{Eq. 13}$$

$$\tau = \tau_f (\text{MPa}) \ \text{for} s_3 < s \qquad \text{Eq. 14}$$

where

$$\tau_b = 2.57\sqrt{f_c'} (\text{MPa}) \qquad \text{Eq. 15}$$

Where $s_1 = 0.15c_0$; $s_2 = 0.15c_0$; $s_3 = c_0$; c_0 is the clear spacing between two ribs of the reinforcing bar. Note that s_1, s_2, and s_3 can be assumed 1.5, 3.5, and 10 mm respectively, in the absence of the bar lug clear spacing.

$$\tau_f = 0.35\tau_b \qquad \text{Eq. 16}$$

In case of spitting failure, Harajli (2009) suggested the following expression to define the necessary stresses and slip points in order draw the bond stress–slip diagram.

$$\tau_{b,c,split} = \gamma\sqrt{f_c'}\left(\frac{c + K_c}{d_b}\right)^{\frac{2}{3}} \leq \tau_b (\text{MPa}) \qquad \text{Eq. 17}$$

$$s_{split} = s_1 e^{3.3 \ln\left(\frac{\tau_{b,c,split}}{\tau_b}\right)} + s_0 \ln\left(\frac{\tau_b}{\tau_{b,c,split}}\right) (\text{mm}) \qquad \text{Eq. 18}$$

$$\tau_{b,p-split} = \tau_{b,c,split}\left(0.5 + 46\frac{E_f n_f t_f}{E_s c n_s}\right) \leq \tau_{b,c,split} (\text{MPa}) \qquad \text{Eq. 19}$$

$$K_c = 56\frac{E_f n_f t_f}{E_s n_s} \qquad \text{Eq. 20}$$

$$K_{cs} = 46\frac{E_f n_f t_f}{E_s c n_s} \qquad \text{Eq. 21}$$

where $\tau_{b,c,split}$ and s_{split} are the splitting bond strength and the corresponding slip for confined or unconfined concrete; $\tau_{b,p-split}$ is the post-splitting bond resistance for confined concrete; K_c and K_{cs} are the confinement parameters; s_0 = 0.15 mm for unconfined concrete; and 0.2 mm for concrete confined with FRP jacket; E_f is the modulus of elasticity of FRP sheets; E_s is modulus of elasticity of steel; n_f is the number of the FRP sheets; t_f is the thickness of the FRP sheets; n_s is the number of developed or spliced tension bars in the section; c is the smaller of side cover, bottom cover, or half the clear spacing between the bars; and α = 0.7 for unconfined and confined concrete.

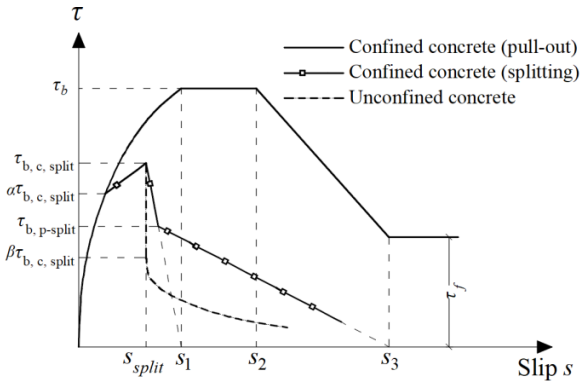


Figure 1. Bond stress–slip response of reinforcing bars in tension (after Harajli *et al.*2006).

The descending portion of the bond stress–slip curve beyond splitting for unconfined concrete can be expressed as follows.

$$\tau = \frac{\beta\tau_{b,c,split}}{\sqrt{\frac{s}{s_{split}}}} (\text{MPa}) \qquad \text{Eq. 22}$$

where β = 0.65 for normal weight concrete where $f_c' \leq 48$ MPa.

3.2 Increase of Bond Strength Due to Increase in Concrete Compressive Strength

Confinement of concrete provides resistance in the transverse direction. An element that is subjected to uniaxial stress will experience strain in the transverse direction in addition to the direction of the applied stress. This phenomenon is referred to as the Poisson's effect. When the applied uniaxial stress is compression, the element tends to expand in the transverse direction. If the element is confined in the transverse direction, the CFRP sheets will restrain the expansion of the element and, hence, create stresses in the direction perpendicular to the applied stress. This will change the state of stress from uniaxial to triaxlial state of stress, which will switch the mode of failure and the stress-strain diagram, and increase the concrete compressive strength.

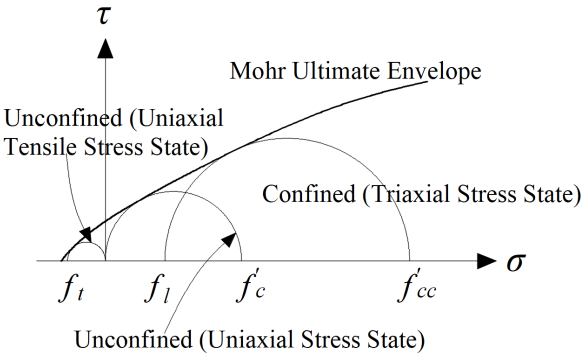


Figure 2. Mohr-Coulomb failure criterion for unconfined and confined concrete.

While some researchers adopted Mohr-Coulomb criterion for concrete failure (Li *et al.* 2003; Mei *et al.* 2001; Gundappa*et al.*, 2001; Domingo *et al.* 2019) (**Error! Reference source not found.**), others used Richart's failure criterion (Xiao and Wu 2000; Li and Hadi 2003; Xiao and Wu 2003; Samaan et al. 1998), which is similar to the Mohr-Coulomb failure criterion. Other failure criteria such as Tsai-Wu and Tsai-Hill, that emphasize on the early failure of the FRP encasement that can be developed by the interaction between the axial and transverse stresses, are also applicable (Mandal *et al.* 2005; Cho *et al,* 2005; Barbero 1998).

The stress-strain curve of confined concrete is dependent on the level of confinement as shown in Figure 3. where $f_c^{'}$ is the specified compressive strength of unconfined concrete,$f_{cc}^{'}$ is specified compressive strength of confined concrete. The above strengths are computed by dividing the peak load by the cross-sectional area of the concrete after deducting the impact of the steel reinforcement. $\varepsilon_{cu}$is the ultimate strain of the unconfined member corresponding to $0.85f_c^{'}$, $\varepsilon_{ccu}$is the ultimate strain of the confined member corresponding to $0.85f_{cc}^{'}$ (lightly confined member) failure strain (heavily confined member) (ACI 440.2R, 2017).

It is important to emphasize that the confined stress-strain diagram should be applied only in the confined section of the beam.

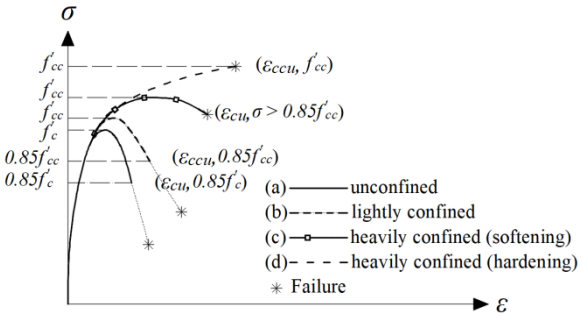


Figure 3. Schematic stress-strain behavior of unconfined and confined reinforced concrete columns (after Rocca *et al.* 2006).

IV. SUMMARY AND CONCLUSIONS

This research investigates the effectiveness of CFRP in increasing the bond between steel reinforcement bars and the surrounding concrete. The author concluded that wrapping concrete beams with CFRP sheets increases the bond between rebars and adjacent concrete because the confinement of concrete improves the bond slip strength. In addition, confining concrete increases the concrete compressive strength, which results in an increase in the bond strength.

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