

# A Comparative Study on Beam Strengthened with Externally Bonded FRP Material

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**Abstract**-In this research work, parametric study is performed to compare the influences of design parameters on flexural moment capacity of externally bonded FRP (Fiber Reinforced Polymer) strengthened beam. Three specific design guidelines: ACI 440.2R-08, ISIS CANADA and TR-55 are considered. In this purpose MS Excel is used to determine the moment capacity for variable parameters specified in the above mentioned three codes. Graphs of capacity against these parameters are plotted to make the comparison well-defined. The three influential parameters that are used in this study are FRP reinforcement, FRP modular ratio, FRP ultimate strength. Most of the parameters under this study influence the flexural moment capacity. The effects of the parameters vary depending on the analysis procedure and the assumption behind of these analyses. Also most of the cases, the effects of these influential parameters increase capacity.

**Index Terms**-Externally bonded FRP material, flexural beam, renovation, strengthening, Fiber Reinforced Polymer, ACI 440.2R-08, ISIS Educational Module, TR-55.

## 1. INTRODUCTION

CONCRETE structures constitute most of the existing civil infrastructure worldwide and they are required to serve a specific function at or above a minimum acceptable performance level. Under the influence of environmental, age and mechanical actions, these infrastructures deteriorate, leading to gradual loss of performance over a period of time. Furthermore, these structures may be under-performing due to changes in the live load requirements, seismic loads and internal defects. Sometimes it is required to rehabilitate these structures rather than replacing them with new ones. Structural strengthening through retrofitting with FRP is one of the many ways aimed at improving the performance level of under-performing structures.

There are several civil engineering codes available which can provide design guidelines for FRP retrofitted structure. However, FRP materials are relatively new to civil engineers and there exists considerable uncertainties in their structural behavior and long term performance. Hence, the design guidelines for FRP applications for structural strengthening tend to be more conservative than usual. Furthermore, individual guidelines propose different criteria for ensuring safety, resulting in conflicting levels of conservativeness. In order to arrive at a common, efficient and rational global basis for design, it is important to evaluate the influence of such criteria on strengthening design.

FRP plate bonding technology was first investigated at the Swiss Federal Laboratory for Materials Testing and Research (EMPA) [Meier, [11]]. MacDonald and Calder [10] studied the behavior of reinforced concrete I-beams strengthened with externally bonded steel plates. The concrete beams were tested under four-point bending. Gemert and Bosch [16] reported durability test results on concrete beams strengthened with epoxy-bonded external

steel plates. The effects of long-term exposure, fatigue, and temperature loading were investigated. Swamy et al. [14], [13] investigated the influence of glued steel plates on the first cracking load, cracking behavior, deformation, serviceability, and ultimate strength of reinforced concrete beams. Forty beams were tested. All test beams had a rectangular cross-section of 225 mm deep by 155 mm wide. Oehlers [12] conducted a series of detailed studies on the failure mechanism of steel plated beams. Test results on concrete beams strengthened with steel plates in the constant moment region showed that the increase in the applied load resulted in a corresponding increase in the curvature of the beam, which could induce flexural peeling. Hussain et al. [8] investigated the influence of a bolted end anchorage on ductility and strength. Bolts were installed to 50 percent of the beam depth. Tests results showed that bolted anchorages could not prevent premature debonding failure. Rutz [15] studied the structural performance of concrete structures strengthened with externally bonded steel plates and subjected to high temperatures. Hankers [7] investigated various failure mechanisms that might occur under repeated loading conditions for concrete members strengthened with externally bonded steel plates. Hankers [7] concluded that members strengthened in flexure with externally bonded steel plates are not sensitive to repeated loading conditions if the plates are properly bonded to the concrete.

Fiber reinforced polymers have different types of fibers including glass (GFRP), carbon (CFRP), and aramid (AFRP), and are available in different grades of tensile strength and modulus of elasticity. Among the different types of current FRP composites for infrastructure applications, CFRP composites are generally the least prone to fatigue failure. The modulus of elasticity and tensile strength of CFRP composites are greater than those of AFRP and GFRPs. GFRP has a lower modulus of elasticity and tensile strength; however, it is more cost-effective in comparison with CFRP.

In this study, the influence of various parameters in flexural strengthening design specifications, as prescribed by three

design guidelines: ACI440.2R-08[1], ISIS CANADA[3] & TR-55[4] has been discussed and analyzed. Analytical models of simply supported beams retrofitted with FRP sheets have been devised to apply different code specifications in order to get nominal strength under existing increased load condition. Parameters involving the nominal strength of the beams have been varied to identify and plot the changes in strength.

In this purpose Microsoft Office Excel has been used to develop the overall calculations prescribed by the three well recognized codes (ACI440.2R-08[1], ISIS CANADA[3] & TR-55[4]) and to draw the respective figures so that the comparisons can be shown in more closely. Here in this study, a simply supported beam with varied parameters (i.e. dimensions, material properties, exposure conditions etc.) has been used. It can be modified for slab or beam with different support condition.

In the strengthening of RC structures, FRP plates or sheets are externally bonded to RC structures using epoxy resins. For the external FRP to be effective in enhancing load-carrying capacity of the structure, effective stress transfer is required between FRP and concrete. The code analyses are based on the fact that the structure and FRP material acts elastically.

The main purpose of this study is to establish a comparative study for governing parameters according to the three code guidelines (ACI440.2R-08[1], ISIS CANADA[3] & TR-55[4]). This comparative study may include the comparison of conservativeness regarding strength or moment capacity.

The influences of different parameters on the strengthening project have been investigated in this study. The variation of these influential effects with respect to the design guidelines used has also been investigated.

Then the same calculation process is done for 10 identical beams with a single varying parameter.

For other parameters the same process is done with different spreadsheets for each of the parameters.

## 2 CODE ANALYSIS

The determination of nominal and ultimate strength for different parametric values is accomplished exactly as specified by the respective code. For this purpose simply supported singly reinforced beams have been considered. The strength influencing parameters that have been varied are FRP ratio  $\rho_f$ ; FRP strength  $f_{fu}$ ; FRP strain  $\epsilon_{fu}$ ; modular ratio of FRP material  $E_f$ .

### 2.1 Analysis according to the ACI 440.2R-08

ACI440.2R has been in its latest form since 2008, superseding the previous 2002 version with significant changes. ACI 440.2R-08[1] has been following the stress distribution diagram of concrete member in bending as per ACI: 318.

*Environmental reduction factor:* The material properties reported by manufacturers, such as the ultimate tensile strength, typically do not consider long-term exposure to

environmental conditions and should be considered as initial properties. Because long-term exposure to various types of environments can reduce the tensile properties and creep-rupture and fatigue endurance of FRP laminates. The material properties used in design equations should be reduced based on the environmental exposure condition. The constituent materials, fibers, and resins of an FRP system affect its durability and resistance to environmental exposure. The environmental reduction factors given in Table 1 are conservative estimates based on the relative durability of each fiber type.

Equations (1) and (2) give the tensile properties that should be used in all design equations. The design ultimate tensile strength should be determined using the environmental reduction factor given in Table 1 for the appropriate fiber type and exposure condition.

$$f_{fu} = C_E f_{fu}^* \quad (1)$$

Similarly, the design rupture strain should also be reduced for environmental exposure conditions.

$$\epsilon_{fu} = C_E \epsilon_{fu}^* \quad (2)$$

TABLE 1  
ENVIRONMENTAL REDUCTION FACTOR FOR VARIOUS FRP SYSTEMS AND EXPOSURE CONDITIONS

Exposure conditions	Fiber type	Environmental reduction factor $C_E$
Interior exposure	Carbon	0.95
	Glass	0.75
	Aramid	0.85
Exterior exposure (bridges, piers, and unenclosed parking garages)	Carbon	0.85
	Glass	0.65
	Aramid	0.75
Aggressive environment (chemical plants and wastewater treatment plants)	Carbon	0.85
	Glass	0.50
	Aramid	0.70

Because FRP materials are linearly elastic until failure (assumption), their design modulus of elasticity for unidirectional FRP can be determined from Hooke's law. The expression for the modulus of elasticity, given in equation (3), recognizes that the modulus is typically unaffected by environmental conditions. The modulus given in this equation will be the same as the initial value reported by the manufacturer.

$$E_f = f_{fu} / \epsilon_{fu} \quad (3)$$

*Failure modes:* The flexural strength of a section depends on the controlling failure mode. The following flexural failure modes should be investigated for an FRP-strengthened section (GangaRao and Vijay [6]):

*Crushing of the concrete in compression before yielding of the reinforcing steel;*

*Yielding of the steel in tension followed by rupture of the FRP laminate;*

*Yielding of the steel in tension followed by concrete crushing;*

*Shear/tension delamination of the concrete cover (cover delamination) and*

*Debonding of the FRP from the concrete substrate (FRP debonding).*

Concrete crushing is assumed to occur if the compressive strain in the concrete reaches its maximum usable strain ( $\epsilon_{cu} = 0.003$ ). Rupture of the externally bonded FRP is assumed to occur if the strain in the FRP reaches its design rupture strain ( $f_f = f_{fu}$ ) before the concrete reaches its maximum usable strain. Cover delamination or FRP debonding can occur if the force in the FRP cannot be sustained by the substrate. Such behavior is generally referred to as debonding, regardless of where the failure plane propagates within the FRP-adhesive substrate region. Away from the section where externally bonded FRP terminates, a failure controlled by FRP debonding may govern. To prevent such an intermediate crack-induced debonding failure mode, the effective strain in FRP reinforcement should be limited to the strain level at which debonding may occur,  $\epsilon_{fd}$  as defined in equation (4)

$$\epsilon_{fd} = 0.83 \sqrt{\frac{f'_c}{n t_f E_f}} \leq 0.9 \epsilon_{fu} \quad \text{in in-lb units}$$

$$\epsilon_{fd} = 0.41 \sqrt{\frac{f'_c}{n t_f E_f}} \leq 0.9 \epsilon_{fu} \quad \text{in SI units} \quad (4)$$

*Assumptions:* the following assumptions are made in calculating the flexural resistance of a section strengthened with an externally applied FRP system:

*Design calculations are based on the dimensions, internal reinforcing steel arrangement, and material properties of the existing member being strengthened;*

*The strains in the steel reinforcement and concrete are directly proportional to the distance from the neutral axis. That is, a plane section before loading remains plane after loading;*

*There is no relative slip between external FRP reinforcement and the concrete;*

*The shear deformation within the adhesive layer is neglected because the adhesive layer is very thin with slight variations in its thickness;*

*The maximum usable compressive strain in the concrete is 0.003;*

*The tensile strength of concrete is neglected; and*

*The FRP reinforcement has a linearly elastic stress-strain relationship to failure.*

While some of these assumptions are necessary for the sake of computational ease, the assumptions do not accurately reflect the true fundamental behavior of FRP flexural reinforcement. For example, there will be shear deformation in the adhesive layer causing relative slip between the FRP and the substrate. The inaccuracy of the assumptions will not, however, significantly affect the computed flexural strength of an FRP-strengthened member. An additional strength reduction factor will conservatively compensate for any such discrepancies.

*Existing substrate strain:* Unless all loads on a member, including self-weight and any pre-stressing forces, are removed before installation of FRP reinforcement, the substrate to which the FRP is applied will be strained. These strains should be considered as initial strains and should be excluded from the strain in the FRP (Arduini and Nanni[2]; Nanni and Gold [9]). The initial strain level on the bonded substrate,  $\epsilon_{bi}$ , can be determined from an elastic analysis of the existing member, considering all loads that will be on the member during the installation of the FRP system. The elastic analysis of the existing member should be based on cracked section properties.

$$\epsilon_{bi} = \frac{M_{DL}(h - kd)}{I_{cr} E_c} \quad (5)$$

*Strain level in FRP reinforcement:* It is important to determine the strain level in the FRP-reinforcement at the ultimate limit state. Because FRP materials are linearly elastic until failure, the level of strain in the FRP will dictate the level of stress developed in the FRP. The maximum strain level that can be achieved in the FRP reinforcement will be governed by either the strain level developed in the FRP at the point at which concrete crushes, the point at which the FRP ruptures, or the point at which the FRP debonds from the substrate. The effective strain level in the FRP reinforcement at the ultimate limit state can be found from equation (6)

$$\epsilon_{fe} = \epsilon_{cu} \left( \frac{d_f - c}{c} \right) - \epsilon_{bi} \leq \epsilon_{fd} \quad (6)$$

Where,  $\epsilon_{bi}$  is the initial substrate strain and  $d_f$  is the effective depth of FRP reinforcement.

*Stress level in the FRP reinforcement:* The effective stress level in the FRP reinforcement is the maximum level of stress that can be developed in the FRP reinforcement before flexural failure of the section. This effective stress level can be found from the strain level in the FRP, assuming perfectly elastic behavior.

$$f_{fe} = E_f \epsilon_{fe} \quad (7)$$

Based on the strain level in the FRP reinforcement, the strain level in the non-prestressed steel reinforcement can be found from equation (8) using strain compatibility

$$\epsilon_s = (\epsilon_{fe} + \epsilon_{bi}) \frac{(d - c)}{(d_f - c)} \quad (8)$$

The stress in the steel is determined from the strain level in the steel using its stress-strain curve

$$f_s = E_s \epsilon_s \leq f_y \quad (9)$$

With the strain and stress level in the FRP and steel reinforcement determined for the assumed neutral axis depth, internal force equilibrium may be checked using equation (10)

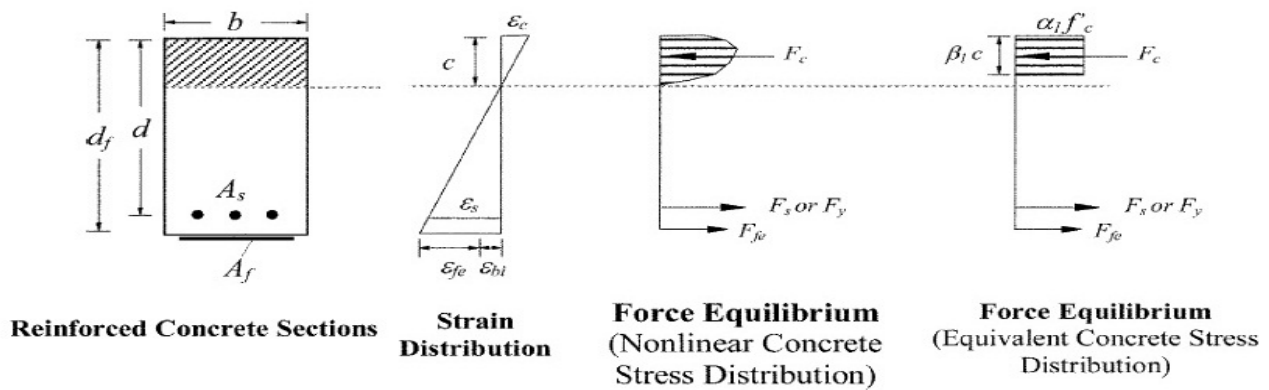


Figure 1: Elastic strain and stress distribution

$$c = \frac{(A_s f_s + A_f f_{fe})}{\alpha_1 \beta_1 f_c b} \tag{10}$$

The terms  $\alpha_1$  and  $\beta_1$  in equation (10) are the parameters defining a rectangular stress block in the concrete equivalent to the nonlinear distribution of stress. If concrete crushing is the controlling mode of failure (before or after steel yielding),  $\alpha_1$  and  $\beta_1$  can be taken as the values associated with the Whitney stress block. If FRP rupture, cover delamination, or FRP debonding occur, the Whitney stress block will give reasonably accurate results. A more accurate stress block for the strain level reached in the concrete at the ultimate-limit state may be used.

The depth to the neutral axis  $c$  is found by simultaneously satisfying equation (5-9) thus establishing internal force equilibrium and strain compatibility. To solve for the depth of the neutral axis  $c$ , an iterative solution procedure can be used. An initial value for  $c$  is first assumed and the strains and stresses are calculated using equation (5-8). A revised value for the depth of neutral axis  $c$  is then calculated from equation (9). The calculated and assumed values for  $c$  are then compared. If they agree, then the proper value of  $c$  is reached. If the calculated and assumed values do not agree, another value for  $c$  is selected, and the process is repeated until convergence is attained.

The nominal flexural strength of the section with FRP external reinforcement is computed from equation (10). An additional reduction factor for FRP  $\psi_f$ , is applied to the flexural-strength contribution of the FRP reinforcement. The recommended value of  $\psi_f$  is 0.85.

$$M_n = A_s f_s \left( d - \frac{\beta_1 c}{2} \right) + \psi_f A_f f_{fe} \left( d_f - \frac{\beta_1 c}{2} \right) \tag{11}$$

## 2.2 Analysis according to ISIS Educational

### Module 4:

ISIS CANADA has given the guideline for design of externally reinforced /bonded FRP laminated beam .In ISIS code the design strain is not limited up to debonding failure, existing substrate strain not deducted for effective strain of fiber and the strength reduction factor are different than ACI method to find out the strength of beam.

*Environment Reduction Factor:* It is not included in this method but the strength reduction factors are high compared to ACI method and so it is mostly compensated.

*Strength Reduction Factor or Resistance Factor*

TABLE 2: STRENGTH REDUCTION FACTOR

Material reduction factor	For building	For building
$\phi_c$ (Concrete)	0.6	0.75
$\phi_s$ (Steel)	0.85	0.9
$\phi_f$ (CFRP)	0.7	0.78
$\phi_f$ (GFRP)	0.6	0.76

Generally the  $\phi_{frp}$  (carbon) = 0.75 and  $\phi_{frp}$  (glass) = 0.5 is considered for the calculation.

*Assumption:* It is assumed that FRPs are perfectly linear-elastic materials. Thus failure of an FRP strengthened section in flexure can be due to FRP rupture or concrete crushing. The ultimate flexural strength for both of these failure modes can be calculated using a similar methodology as that used for steel-reinforced sections. Hence the following additional assumptions are required:

*Plane sections remain plane;*

*Perfect bond exists between concrete and steel reinforcement and between concrete and FRP reinforcement;*

*Adequate anchorage and development length is ensured for the FRP reinforcement;*

*FRPs are linearly elastic to failure;*

*Concrete compressive stress-strain curve is parabolic and concrete has no strength in tension;*

*Initial strain in the section at the time of strengthening can, in most cases, be ignored.*

In addition steel is treated as elastic-perfectly plastic, with strain hardening neglected and concrete is treated using the concept of equivalent stress block.

*Failure modes:* There are four potential flexural failure modes for externally-strengthened reinforced concrete flexural members:

*Concrete crushing before yielding of reinforced steel;*

*Steel yielding followed by concrete crushing;*

*Steel yielding followed by FRP rupture and*

*Debonding of the FRP reinforcement at the FRP/concrete interface.*

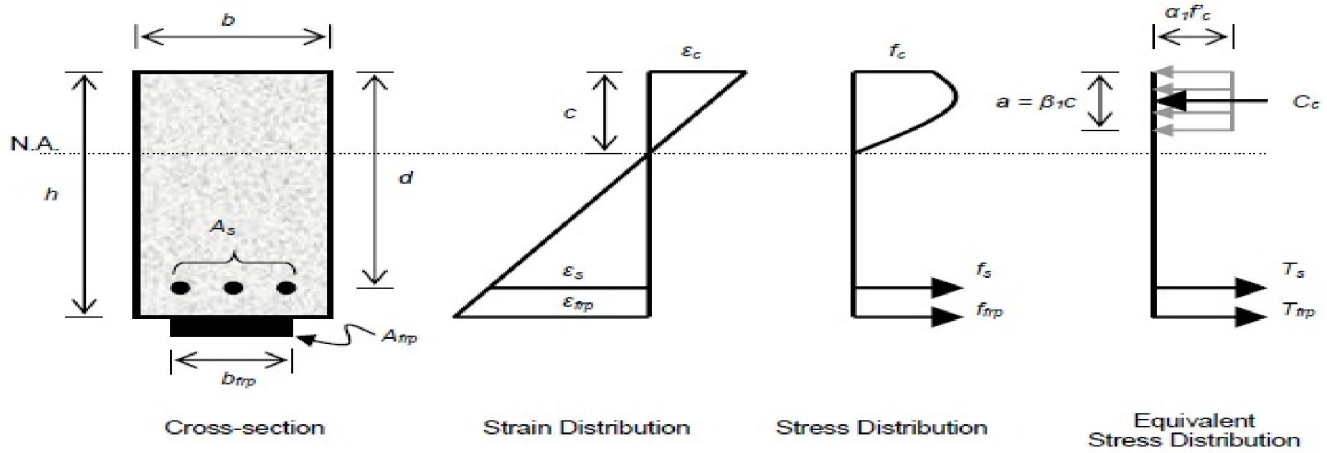


Figure 2: Stress strain profile of a beam strengthened in flexure with externally bonded FRP material.

It is not clear at the outset of a design or analysis which of the above failure modes will govern. Thus an assumption is made and the failure mode is checked.

If the assumption is incorrect, another failure mode is assumed and the analysis is repeated. In usual cases, it is assumed that the fourth failure mode, FRP debonding, will not occur and can be ignored.

Equilibrium of internal forces requires that three stress resultants (concrete in compression  $C_c$ , steel in tension  $T_s$ , FRP in tension  $T_{frp}$ ) sum to zero. Thus

$$C_c = T_s + T_{frp} \tag{12}$$

The stress resultants can be determined using the following expressions;

$$C_c = \phi_c \alpha_1 \beta_1 f_c' b c$$

$$T_s = \phi_s A_s f_s \text{ with } f_s \leq f_y \tag{13}$$

$$T_{frp} = \phi_{frp} A_{frp} E_{frp} \epsilon_{frp} \text{ with } \epsilon_{frp} \leq \epsilon_{frpu} \tag{14}$$

Equation (11) can be used in combination with equation (12-14) and strain compatibility as illustrated in figure 2 to determine the depth of neutral axis. Thus the moment capacity can be determined as;

$$M_n = T_s \left(d - \frac{a}{2}\right) + T_{frp} \left(h - \frac{a}{2}\right) \text{ where } a = \beta_1 c \tag{15}$$

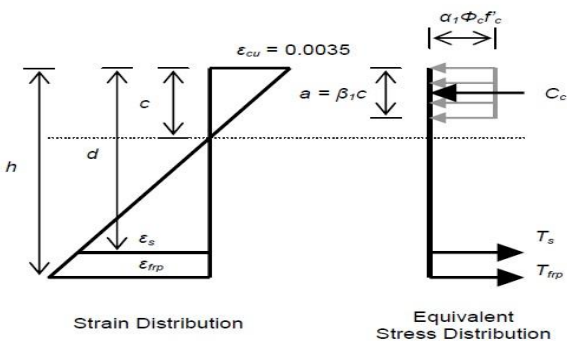


Figure 3: Assumed strain distribution at failure.

### 2.3 Analysis according to The Concrete Society TR-55

Bonding FRP to the tension face increases the flexural strength of concrete elements. Failure of the element may then occur as a result of various mechanisms, as follows:

- Concrete crushing or FRP rupture
- FRP separation due to end peel or debonding.

Comparison of the design ultimate moment  $M_u$ , with the balanced moment of resistance  $M_{r,b}$ , predicts the likely mode of failure. The moment  $M_u$  can be calculated using elastic or plastic methods of analysis, although it is common practice to adopt the former. The balanced condition is assumed to occur when the concrete and the FRP reach their ultimate strains simultaneously, as shown in Fig. 3. Equation (16), obtained by taking moments about the bottom face, can be used to calculate  $M_{r,b}$ .

$$M_{r,b} = \frac{0.67}{\gamma_{mc}} f_{cu} 0.9bx \left[ z + (h - d) \right] - \frac{f_y}{\gamma_{ms}} A_s (h - d) \tag{16}$$

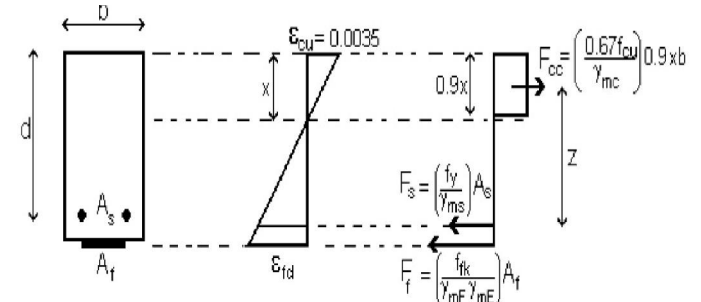


Figure 4: Stress and strain distributions at balanced condition.

If  $M_u > M_{r,b}$ , failure occurs due to concrete crushing and the area of FRP can be determined using standard stress-strain curves for concrete and steel reinforcement by assuming that the FRP reinforcement has a straight line response to failure as shown in Figure 4. The area of FRP is estimated using the following expression

$$A_f = \frac{F_f}{f_f} \tag{17}$$

Where  $F_f$  is the tensile force in the FRP, and  $f_f$  is the stress in the FRP given by

$$f_f = E_{fd} (\varepsilon_{cft} - \varepsilon_{cit}) \quad (18)$$

In which  $E_{fd} = \frac{E_f}{\gamma_{mE}}$  is the elastic modulus of FRP,  $\varepsilon_{cft}$  is the final tensile strain in the concrete at the FRP/concrete interface, and  $\varepsilon_{cit}$  is the initial tensile strain in the concrete at the FRP/concrete interface calculated on the basis of an elastic analysis of the cracked section.

If  $M_u < M_{r,b}$ , failure theoretically occurs due to FRP rupture. However, laboratory tests show that FRP rupture is a rare event and plate separation due to debonding is more likely. The critical value of strain is used to calculate the design stress in the FRP.

### 3 RESULTS AND DISCUSSION

For the final step of the comparative study, the obtained capacities for 10 beam specimens for various parameters according to the 3 codes under this study are plotted in plain graphs. Every parameter has its own graphical representations. Since each graph contains 3 plotted curves for 3 Codes, the variations of the 3 Codes have been shown through these graphs.

#### Parametric Study

##### Case I. FRP Reinforcement Ratio ( $\rho_f = \frac{A_f}{bd}$ )

Figure 5 shows the variation of flexural moment capacity for the changes in FRP application ratio.

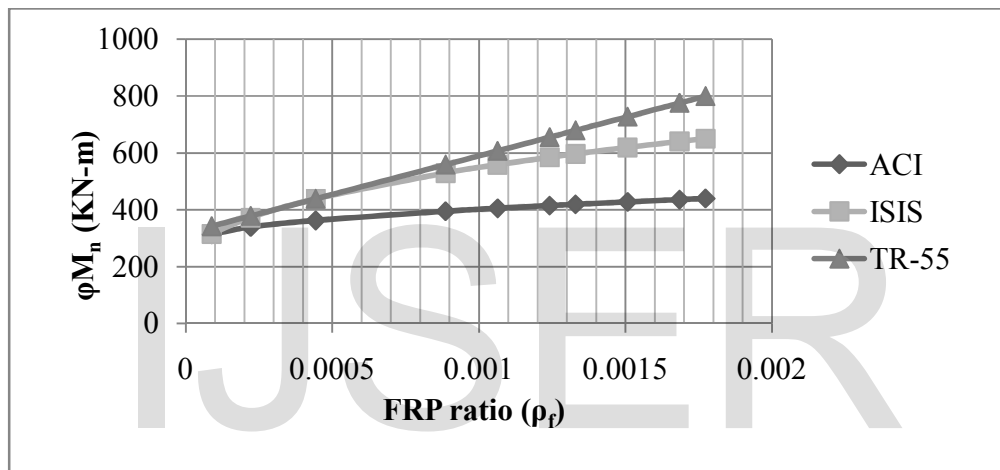


Figure 5 Moment Capacity versus FRP Ratio

It shows that the range of FRP reinforcement ratio  $\rho_f$  is within 0.2% to 2.5% and thus obtained moment capacities vary widely. For small  $\rho_f$  the change in capacities for the three codes are not significant, but for larger  $\rho_f$  ratios the capacity difference is wide. It shows that ACI 440 code provide the more conservative result and TR-55 the least.

Under the same conditions ( $\rho_s = 0.4\%$ ,  $f_y = 415\text{Mpa}$ ,  $f'_c = 30\text{Mpa}$ ,  $f_{fu} = 2800\text{Mpa}$ ,  $E_f = 165\text{Gpa}$ ) ISIS dictates failure by concrete crushing after  $\rho_f$  crosses 0.0089%, while all other codes determine failure by FRP rupture. Canadian code imposes strong F.S (Concrete Compressive Stress Block Factor  $\alpha_1$  and Concrete Compressive Stress Block Factor  $\beta_1$ ) on concrete compressive strength  $f'_c$  more than the other codes resulting in failure by concrete crushing abundant.

##### Case II. FRP Modular Ratio $E_f$ (Gpa)

Figure 6 shows the variation of moment capacity with changes in FRP modular ratio  $E_f$ .

It shows that TR-55 predicts the lowest capacity than ACI 440 and ISIS. Hence TR-55 is more conservative in the context of  $E_f$ . ACI prediction goes next to it. ISIS shows the least conservativeness for FRP modular ratio  $E_f$ .

The range of  $E_f$  has been taken from 13 Gpa to 640 Gpa. At the smaller values of  $E_f$  the differences in capacities for the three codes are small. With the increase in  $E_f$  the change of capacities for the codes varies widely.

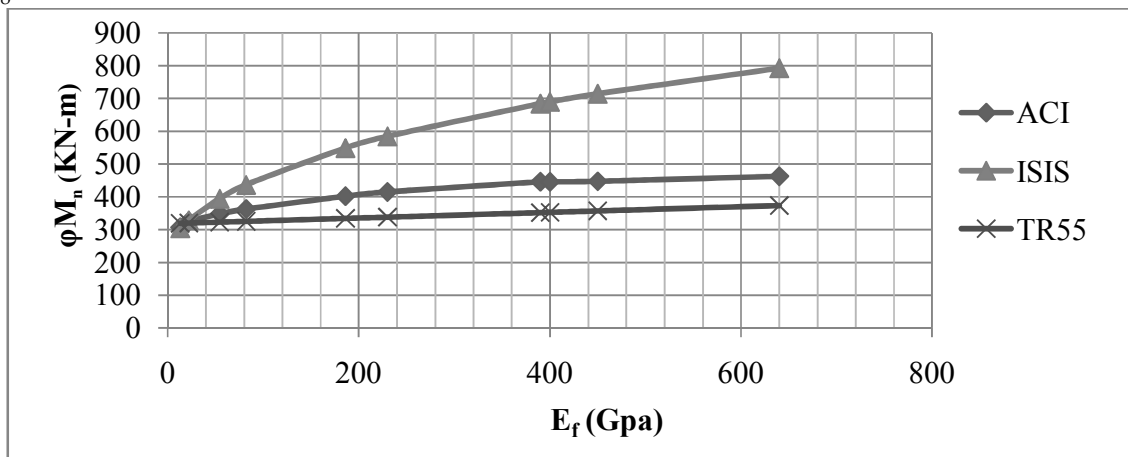


Figure 6: Moment Capacity versus FRP Modular Ratio

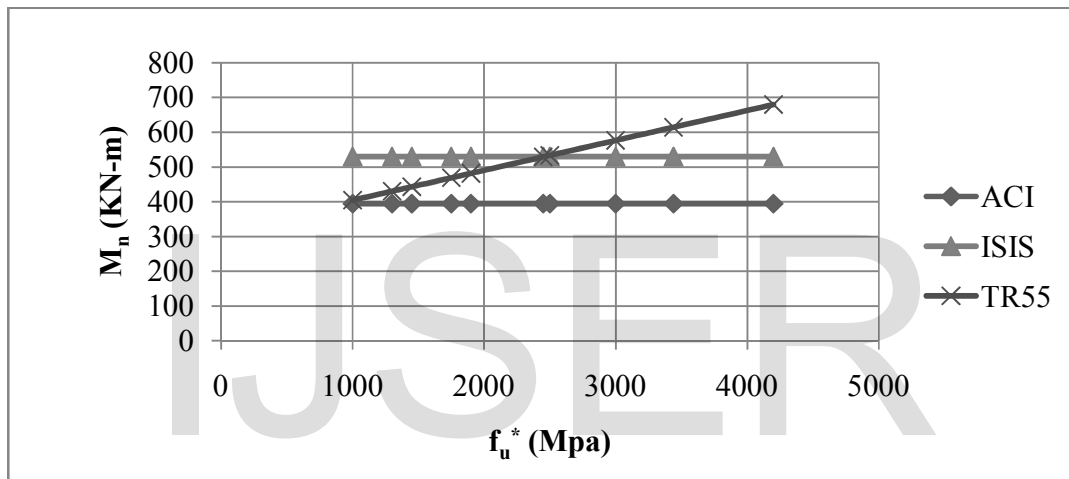


Figure 7: Moment Capacity versus FRP Ultimate Strength

### Case III. FRP Ultimate Strength $f_{fu}^*$ (Mpa)

The variation of flexural moment capacity for the changes of FRP ultimate strength from 1000 Mpa to 4200 Mpa has been shown in figure 7.

It shows a clear distinction among the three codes; ACI 440, ISIS and TR-55. ACI and ISIS have a straight line which means that moment capacities are independent of FRP ultimate strength  $f_{fu}^*$ . This type of graphs might have resulted from the fact that both the codes follow strain compatibility method. Hence the analysis procedures do not employ FRP ultimate strength. Both of these codes use  $f_{fu}^*$  to limit the stress condition as a limiting value for the determination of failure condition. On the other hand TR-55 involves both stress and strain compatibility (Eq.3-18) resulting in a steep sloped straight line. Hence the flexural capacity increases steadily with  $f_{fu}^*$ . From the aspect of conservativeness ACI is most conservative and ISIS and TR-55s conservativeness are next to it.

## 4 CONCLUSIONS

From the detailed study of the three widely recognized codes (ACI 440, ISIS and TR-55) it can be said that most of the parameters studied have beneficiary influence on moment capacity. These influences of the parameters largely depend on the analysis procedure and the assumptions behind these analyses. Hence the effects of some parameters can be used in a beneficiary way for an old and failing beams retrofitting while others cannot be. From the study in this research work the following conclusions can be drawn:

Application of FRP at the tension side generally increases flexural moment capacity of a beam. Increase in FRP application ratio,  $\rho_f$  increases moment capacity significantly. In this context, ACI 440 shows quite conservativeness than ISIS and TR-55.

The modular ratio  $E_f$  acts also congenially for the retrofiting. Hence if a FRP sheet has a higher modulus of elasticity it will add to the moment capacity upon application externally than a FRP sheet with lower modulus of elasticity. Though, TR-55 show less dependency on it and ISIS shows the most influence.

FRP ultimate strength  $f_{fu}^*$  which is provided by the manufacturer has significance in retrofiting if the design is made following British Standard (TR-55). The other codes depend on it only to limit the stress developed in FRP.

Design procedures and the guidelines followed play an important role in the resulting capacity which can be verified through practical experimentations.

The surrounding environmental condition affects the resulting moment capacity. Hence the analytical result may vary with the practical capacity depending on the type of region the structure is located.

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