The Need of a Unified Model for Strength Analysis of Strengthened Concrete Columns

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ABSTRACT

The current paper reports selected findings from a research programme carried out at the University of Southampton into the fundamental understanding of the effectiveness of FRP confinement, when applied to circular concrete columns as a means of improving strength. The paper shows that predictions from the empirical formulae-based design guidelines recommended in *The Concrete Society Technical Report No. 55* (the most widely used design standard in the UK) and *ACI 440.2R-02* and *ACI 440.2R-08* (the most widely used design standards in the USA) are unreliable. By comparing predictions from the code formulae for a database of test results, extracted from the published literature, the paper shows that the formulae fail to provide correct failure load. The paper shows that the models do not take into account of the effects due to: for instance, low confinements in high strength concrete, rupture strain of FRPs, change in the FRP rupture strain in jackets with multiple layers, and localised FRP failures likely to happen in large diameter columns, etc. In particular, it is shown that, although predictions form the *Concrete Society* model generally agree with the test results used to derive the original empirical formulae, it does not provide accurate predictions for failure load of new test specimens. The paper also identifies key design parameters, which may be incorporated in an accurate unified model to predict strength of FRP-confined columns.

INTRODUTION

Fibre reinforced polymers (FRP) have a higher ultimate strength and strain than steel, and hence have a strength to weight ratio significantly higher than that of steel. The high strength to weight ratio together with its suitability in hostile environments, the FRP materials are considered to have great potentials in the construction industry although cost of FRP is typically a few times the cost of steel on a cost/unit-force/unit length basis [1]. Carbon, Glass and Aramid FRPs have been used widely, however, the inappropriate use of the materials, in particular its use as replacement to steel with the view to replacing one material with another means the applications so far only achieved a limited success. In practice, it is necessary to exploit high strength/strain characteristics of the materials, and one notable successful applications of FRPs in construction industry has been its use for repair and strengthening of concrete structures [2].

Strengthening of concrete structures by using externally-bonded FRP systems have been successfully used in a number of applications where an increased strength capacity is needed, or a change (an increase) of loading, or when the structure needed a repair after a damage. Strengthening with externally-bonded FRPs offers advantages over traditional techniques (e.g. steel plate bonding, section enlargement, external post-tensioning, etc.) since the FRPs are light weight, relatively easy to install, and also due to its non-corrosive characteristics. Strengthening with externally bonded FRPs has reputation as a “safe” method in the sense that the structure will not be made worse by the repair. Flexural and shear strengthening of concrete beams and slabs can be achieved by bonding pultruded FRP strips or by placing FRP bars into slots cut in the cover region of concrete. Design guidelines and standard practices for flexural and shear strengthening are now well established, and a large number of structures (e.g. buildings and bridges), all over the world, were successfully strengthened.

A large amount of FRP repair applications has been on strengthening of circular concrete columns with externally connected FRP jackets. Wrapping a circular column with FRP composites (**Fig. 1**) has potentials to increase the strength and ductility. In the USA and Japan, a large amount of researches has been carried out to exploit the use FRP wraps as a cost-effective retrofitting/strengthening method for columns. Typically, FRP jackets formed by wet lay-up arrangements are used in practice.

Although FRP-jackets strengthening of reinforced concrete columns has worldwide applications, it should be appreciated that there is no widely accepted or validated design method. Strengthening is mostly carried out using largely empirical design guidelines, determined using limited databases of test results. For instance, the current paper shows that the empirical formulae recommended in the most widely used design codes in the UK and USA (*The Concrete Society Technical Report No. 55* [3] and *ACI 440.2R-02* [4] and *ACI 440.2R-08* [5] respectively) fail to predict failure load of a set of strengthened column specimens randomly chosen from the published literature. The paper shows that the design guidelines more often than not fail to accurately predict failure load of new test specimens, which were not included into the databases used to determine the respective original empirical model. There is no certain analysis of the rupture strain of FRPs, effect of the diameter of specimens, concrete grade strength etc. To the best of the authors' knowledge, at the time of writing (April 2013), any of the strengthened columns has not yet subjected to an extreme load event such as a major earthquake where the designs may be seriously tested. The paper presents selected results, discussing the inherent limitations associated with the current design guidelines.

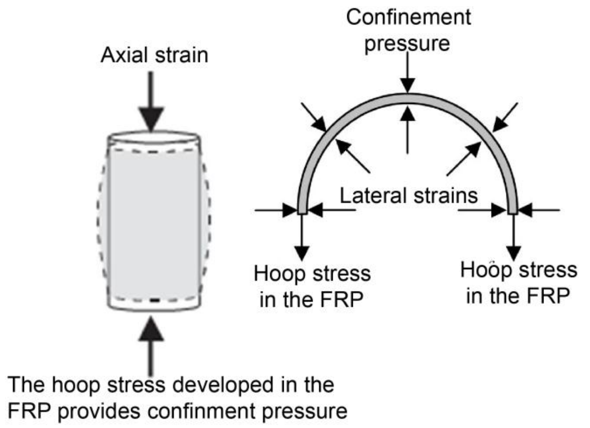


Fig. 1. Externally bonded FRP jackets can be used to provide a confinement pressure

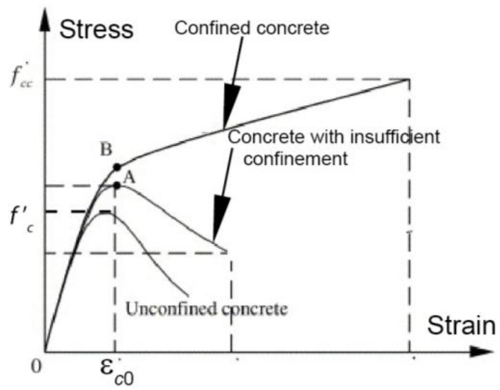


Fig. 2. Stress–strain relationship of confined concrete

FRP STRENGTHENING OF CONCRETE COLUMNS

Concrete columns in existing structures can be upgraded in axial, flexure and shear by using external FRP jackets; strengthening also enhances the ductility. The FRP jacket resists lateral expansions caused by the axial load, resulting in a confining stress in the concrete core (**Fig. 1**); this enhances both strength and strain capacities of concrete. A strengthened column is therefore having a high axial and shear load capacities with an improved ductility. A strengthened column may be failed due to one of 1. tensile rupture of FRP 2. failure of FRP jacket at lap joint 3. shear failure 4. local deboning of FRP jacket 5. concrete compression failure.

Based on results of lab and field tests, the current design codes assume that FRP rupture as the critical failure mode. The FRP rupture can occur either 1. when the stress in the FRP reaches ultimate hoop stress of the material 2. local failure of the resin when the cracking in the concrete causes high stress concentration etc. However, only the failure takes place at the ultimate hoop strain of FRP is considered in the existing design, despite a standard test method has not yet been recommend in the literature to estimate hoop failure strain. This paper critically analyses the feasibility of the design guidelines recommended in three of the currently widely used design codes: *The Concrete Society Technical Report No. 55* and *ACI 440.2R-02* and *ACI 440.2R-08*.

FRP–CONFINED CONCRETE

Stress–Strain relationship

In the design of strengthened columns, it is necessary to consider the effect of the confinement provided by the FRP. Although the effect of FRP confinement has been studied extensively, essentially using experimental results of small cylinders (usually 150 mm x 300 mm), none of the models reported in the literature has yet been accepted by a wider community of researchers and/or practitioners. It should be noted that, for instance, the models do not explicitly model the relationship between the confinement pressure and the axial strain. The design code formulae usually recommend simplified empirical models, which were mostly validated by using the same original database that used to derive the unknown fitting parameters in the respective empirical formulae. It should also be appreciated that the confinement pressure provided by the FRP constantly increases with the applied axial load, and hence the mechanics of confined concrete cannot be accurately modelled using the classical work of material characterisation of hydrostatically confined concrete in which case the confinement pressure remains constant during the increase in applied axial load.

It is expected that under low confined pressures, the behaviour of concrete is similar to that of unconfined concrete. Once the axial strain increases above the strain relating to the peak stress of unconfined concrete (typically ~0.002 [3]), significant lateral strains will be developed due to the Poisson's Ratio's effects and also due to the decrease in the stiffness. Subsequently, the resultant confinement provided by the jacket will be fully-active. Test results reported in the literature show that, when concrete is confined by a FRP, the axial stress increases approximately linear manner with the increase of the applied axial strain. Therefore, in the previous researches, the axial stress–strain response of confined concrete was generally assumed to be that of unconfined concrete at stresses below the peak stress of unconfined concrete, and an approximately linear variation thereafter (**Fig. 2**). The stiffness of confined-concrete depends on that of the FRP jacket [3], and if the confinement provided by the FRP is insufficient then the axial-stress may decrease (**Fig. 2**).

Modelling the effect of FRP confinement

The experimental results reported in the literature and the available simple empirical formulae provide useful insight into the stress–strain relationship of FRP-confined concrete. However, the validity of the results is limited to the specific test specimen used and the parameters chosen in each case. Therefore, prediction for the design load of a new column is unreliable. There is a need for a unified model which takes account of the interaction between the confinement pressure and the axial strain, instead of empirical formulae developed by matching test results of a finite number of cylinders.

Stress – Strain Model for FRP-Confined Concrete

Under an increasing axial strain, unlike a steel jacket, which is yield, an FRP jacket applies continuously increasing confining pressure till the failure of the FRP. The amount of confinement provided by the jacket depends on the lateral dilation of concrete, which in turn depends on the confining pressure. Despite numbers of design-oriented or analysis-oriented empirical/semi-empirical models have been reported in the literature to characterise the axial strain–confinement pressure relationship, there is little independent validation on the accuracy. In particular, it is difficult to model the localised effects; for instance, it is difficult to differentiate whether the failure occurs at the hoop rupture strain of the jacket or due to a localised failure initiated at a crack developed in the epoxy. Similarly, it is difficult to know whether the spatial pressure distribution over the cross section is uniform since the distribution aggregates and local microcracks will influence lateral strain distribution.

A comprehensive review and assessment of all published FRP-confined concrete models is beyond the scope of the paper, and instead the readers are referred to review papers such as Jiang and Teng [6]. The objective of the current paper is to test the accuracy of the predictions from the design codes against a database test results, randomly chosen from the published literature; limitations associated with the empirical models are discussed below. It should be noted that since the study focuses on modelling the basic mechanics of the FRP confinement, only the simplest form of application – axial compressive strength of non-slender columns under monotonic loads – are considered below.

DESIGN CODES

The first design code for FRP strengthening concrete structures, *The Concrete Society Technical Report 55* (1st edition) was published in 2000. Subsequently, guidance documents have been published in various countries, including the USA, Japan and Canada. In 2002, the *Canadian Standard Association* published the first national code [7]. In the UK, *Highways Agency* published guidance for strengthening bridges using FRP (BD 85/08) [8]. The ACI design standard ACI440.2R-02 published in the early 2000s [4] and it has been updated with a new edition – ACI 440.2R-08 [5]. Design guidelines were also developed by *Japan Society of Civil Engineers* [9], and *ISIS Canada Research Network* [10]. In the UK, advice on designing of adhesively bonded joints for FRP materials may be found in the *EUROCOMP* design code and handbook [11].

The concrete Society – Technical Report No. 55 [3]

The 3rd edition of *Concrete Society Technical Report 55 – Design guidance for strengthening concrete structures using fibre composite materials* (2012) – is the current UK industry standard on FRP strengthening. This design code is recommended to use in conjunction with the relevant Eurocodes (e.g. BS EN 1990 – Basics of Structural Design [12], BS EN 1991 – Actions on Structures [13], BS EN 1992 – Design of Concrete Structures [14], etc.).

Based on the empirical model developed by Teng et al. [15], *The Concrete Society Technical Report 55* recommends to determine the ultimate stress of FRP-confined concrete ($f_{cc}^{'}$) using the following equation.

$$f_{cc}^{'} = f_c^{'}\left\{1 + 5.25\left(\rho_K - 0.01\right)\rho_\varepsilon\right\} \tag{1}$$

Where $f_c^{'}$ is the unconfined concrete strength (**Fig. 2**). The stiffness ratio (ρ_K) and the strain ratio (ρ_ε) can be determined as follows (Eqs. 2* and 3 respectively).

$$\rho_K = \frac{2E_f\, t_f}{\left(\frac{0.85 f_{ck}}{\varepsilon_{c0}}\right)\phi} \tag{2}$$

where f_{ck} = characteristic unconfined concrete cylinder strength

ε_{c0} = axial strain in unconfined concrete at peak stress [3]

ϕ = diameter of the column; E_f = Young's modulus of FRP; t_f = thickness of FRP wrap

(*Note: it should be noted that Eq. 2 uses the characteristic strength of the unconfined concrete with a strength reduction factor of 0.85; however, in the analysis below the unfactored mean strength of unconfined concrete (f_m) was used to ensure a better comparison with test results.)

The strain ratio is determined as:

$$\rho_\varepsilon = \varepsilon_{h,rup} / \varepsilon_{c0} \tag{3}$$

The hoop rupture strain of the FRP jacket ($\varepsilon_{h,rup}$) is assumed to be 0.6 times the uniaxial tensile strength of the FRP fabrics, which is usually determined from standard coupon tests [16] .

ACI 440.2R-02 [4]

The ACI 440.2R-02 design guidelines for externally bonded FRP systems are recommend to be used in conjunction with the relevant ACI concrete codes (e.g. ACI 318-08 [17]). ACI 440.2R-02 recommends an empirical formula (Eq. 4), originally developed by Mander et al. [18] to model the confinement provided by steel jackets, to determine of the ultimate confined strength of FRP–confined concrete. The code recommends this empirical formula for the analysis of FRP-confined concrete based on the study of Spoelstra and Monti [19], where the model predictions reasonably agreed with a small database of test results of small-size concrete cylinders (diameter of the largest tested cylinders = 200 mm). Nevertheless, the accuracy of this model is dubious due to the fundamentally different material behaviour of the two materials: a FRP jacket applies a continuously increasing confining pressure whereas a constant confining pressure provided by a yielded steel jacket.

$$f'_{cc} = f'_c \left[2.25 \sqrt{1 + 7.9 \frac{f_l}{f'_c}} - 2 \frac{f_l}{f'_c} - 1.25 \right] \tag{4}$$

The confining pressure (f_l) may be determined as:

$$f_l = \frac{\rho_f \, \varepsilon_{fe} \, E_f}{2} \tag{5}$$

where the maximum effective hoop strain in the FRP (ε_{fe}) may be assumed as the lowest of 0.004 or 0.75 times the ultimate tensile strain obtained from coupon tests of FRP (ε_{fu}). The FRP reinforcement ratio (ρ_f) can be calculated using (Eq. 6) by taking account of the thickness of the FRP (t_f), number of FRP layers applied (n), and the diameter of the column (ϕ).

$$\rho_f = \frac{4 n t_f}{\phi} \tag{6}$$

ACI 440.2R-02 [5]

The current edition of ACI 440 code (ACI 440.2R-02 [5]) recommends to determine the ultimate stress of FRP-confined concrete (f'_{cc}) using a different empirical formula developed by the same research team that developed the empirical formulae incorporated in the Concrete Society code. In the current formula (Eq. 7), f'_{cc} is determined by substituting the confining pressure (f_l) determined using the same formulae as the previous version of the code (Eq. 5 and 6) in a different empirical model (Eq. 7). It should be noted that the structure of this empirical formula of Lam and Teng (2003) [20] is significantly different to that of the model incorporated in the *Concrete Society Technical Report 55* (Eq.1). The present authors believe that the latter empirical formula (Eq. 1) was derived using a relatively large database of test results than that used to determine the former (Eq. 7).

$$f'_{cc} = \psi_f \, 3.3 \, f_l \tag{7}$$

where ψ_f, strength reduction factor for FRP = 0.85; and the confining pressure (f_l) may be determined from Eq. 5. However, a different value, 0.6 times the ultimate tensile strain obtained of coupons (ε_{fu}), is recommended as the design hoop strain of the FRP (ε_{fe}).

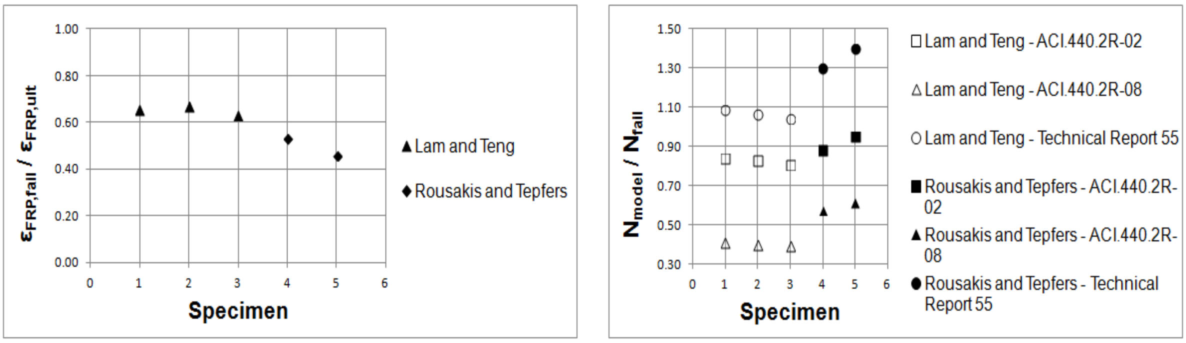
RESULTS: PREDICTIONS FROM THE MODELS

The respective empirical formula recommended in each design codes was determined by matching a limited number of test results of small (mostly 150 x 300 mm) concrete cylinders. Although, the formulae empirically take account of factors such as the effect of column size on the effectiveness of the FRP confinement, by comparing the model predictions for a database of test results, randomly extracted from the published literature (Table 1), it will be shown below that the formulae do not provide accurate or consistent results. In particular, although the predictions form *The Concrete Society* model agree well with the test results used to derive the empirical formula [15], it does not provide accurate predictions for failure load of new specimens.

Predictions from the empirical models for new test specimens

Fig. 3a shows the ratio between FRP strain at failure ($\varepsilon_{FRP,fail}$) and the ultimate strain determined from the uniaxial tensile tests of FRP coupons ($\varepsilon_{FRP,ult}$) for three test specimens chosen from Lam and Teng

[16] (these specimens were included in the Teng et al. database [15]) and two specimen randomly chosen from elsewhere in the literature (Rousakis and Tepfers [21]). The test cylinders were 150 mm (diameter) by 300 mm (height), and strengthened with 2 layers of CFRPs (Table 1). From **Fig. 3a**, it can be seen that the effective FRP strain ratio is ~0.65 for the three specimens chosen from the Lam and Teng study [16] whereas that of the other specimens [21] varies between 0.53 and 0.46. **Fig. 3b** shows the ratio between the ultimate axial load (N_{model}) predicted from the three design code models respectively and the observed failure load (N_{fail}) of the same test specimens. It should be noted that the results determined from the Concrete Society and ACI models are shown in the figure. It can be seen from **Fig. 3b** that the predictions from the Concrete Society model agree reasonably well with the test results of Lam and Teng [16] specimens with a N_{model} / N_{fail} ratio of ~ 1.0–1.1. However, the results show that the model significantly overestimates the design load of other two specimens. Similar results were noted for a large number of test specimens randomly chosen from different test programmes reported in the literature (due to space limitations all results cannot be shown in the current paper). ACI 440.2R-02, in general, provides relatively consistent and accurate results with a load ratio of ~0.8–0.95 whereas ACI 440.2R-08 underestimate the design load by a large amount.



(a) **(b)**

Fig. 3. Predictions from *The Concrete Society* model are not accurate for the failure load of new specimens **(a)** $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ ratio **(b)** N_{model} / N_{fail} ratio

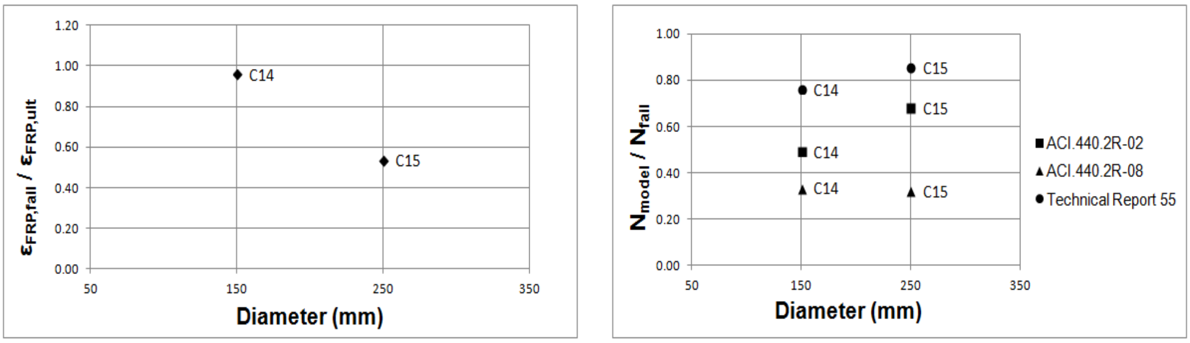
Columns with different diameters

The design code formulae are based on the assumption that a uniform spatial distribution of the lateral strain. Although it is appropriate to assume a uniform "average" lateral strain over a small cross sectional area, the local distribution of lateral strain in a large column can be very complex; for instance, due to the material heterogeneity and the complex and unknowable distribution of aggregates and voids means that the distribution of lateral strain may not be uniform. Similarly, it is difficult to know whether a known confining pressure in a large and small column respectively causes the same lateral strain distribution. **Fig. 4** investigates the effect of the diameter of the columns on the strength enhancement. **Fig. 4a** and **Fig. 4b** show the ratios $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ and N_{model} / N_{fail} respectively for two strengthened cylinders of with diameters 150 mm and 250 mm respectively, reported in the study of Silva and Rodrigues [22]. It can be seen from **Fig. 4a** that the FRP strain ratio $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ decreases to ~0.96 from 0.53 when the diameter of the cylinder increases from 150 mm to 250 mm. This may be attributed to two reasons. 1. The size effects in FRP materials 2. localised FRP failure in the large specimen. Subsequently, the N_{model} / N_{fail} ratio increases from ~0.5 to ~0.75 (Concrete Society model) and ~0.7 to ~0.85 (ACI 440.2R-02) when the diameter increases from 150 mm to 250 mm, although the predictions from ACI 440.2R-08 remains mostly the same. Similar results were noted for a large number of test specimens chosen from different test programmes reported in the literature (due to space limitations all results cannot be shown in the current paper). The results illustrates that the failure strain in the FRP jackets depends on the size of the column and hence there is a need for a better understanding of the failure mechanism of FRPs. A large study, considering test columns with diameters larger than 250 mm, is currently being undertaken and the results will be published in due course.

Table 1. Details of the Test specimens

Reference	Original column identification	Assigned column notation	$\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$	N_{model} / N_{fail}		
				ACI.440. 2R-02	**ACI.440. 2R-08**	**Technical Report 55**
Rousakis and Tepfers [21]	20c1L1-2t	C1-$C_1L_{300}\varnothing_{150}f_{25}$	0.53	1.08	0.48	1.31
	20c1L1-3t					
	20c1L2-1t	C2-$C_2L_{300}\varnothing_{150}f_{25}$	0.50	0.96	0.67	1.47
	20c1L2-2t					
	20c1L3-1t	C3-$C_3L_{300}\varnothing_{150}f_{25}$	0.34	0.96	0.86	1.75
	20c1L3-2t					
	40c1L2-1t	C4-$C_2L_{300}\varnothing_{150}f_{47}$	0.50	1.01	0.47	1.25
	40c1L2-2t					
	80c1L2-1t	C5-$C_2L_{300}\varnothing_{150}f_{71}$	0.40	1.14	0.40	1.29
	80c1L2-2t					
Lam and Teng [16]	C1-2	C6-$C_1L_{305}\varnothing_{152}f_{36}$	0.65	0.96	0.28	1.02
	C1-3					
	C2-1	C7-$C_2L_{305}\varnothing_{152}f_{36}$	0.65	0.83	0.40	1.07
	C2-2					
	C2-3					
	C3-1	C8-$C_3L_{305}\varnothing_{152}f_{36}$	0.59	0.72	0.47	1.08
	C3-2					
	C3-3					
	G1-1	C9-$G_1L_{305}\varnothing_{152}f_{39}$	0.93	0.84	0.25	0.87
	G1-3					
	G2-1	C10-$G_2L_{305}\varnothing_{152}f_{39}$	0.87	0.73	0.37	0.97
	G2-2					
	G2-3					
Silva and Rodrigues [22]	EE-30-A	C11-$G_2L_{300}\varnothing_{150}f_{27}$	0.93	0.49	0.33	0.75
	EE-30-C					
	EE-45-A	C12-$G_2L_{450}\varnothing_{150}f_{27}$	0.94	0.47	0.32	0.73
	EE-45-C					
	EE-60-A	C13-$G_2L_{600}\varnothing_{150}f_{27}$	0.80	0.52	0.35	0.80
	EE-60-C					
	EE-75-A	C14-$G_2L_{750}\varnothing_{150}f_{27}$	0.96	0.49	0.33	0.76
	EE-75-C	C15-$G_2L_{750}\varnothing_{250}f_{27}$	0.53	0.68	0.32	0.86

(*Note: The notation of the columns were chosen to show key parameters of the concrete/FRP strengthening system. 1st letter: C (CFRP)/ G (GFRP) and the subscript represents the number of FRP layers used; 2nd letter, *L*: height of the column (subscript represents the height in millimetres); 3rd letter, ϕ : diameter of the column with subscript gives the diameter in millimetres; and 4th letter, *f* : mean strength of unconfined concrete with subscript representing the value in N/mm^2)

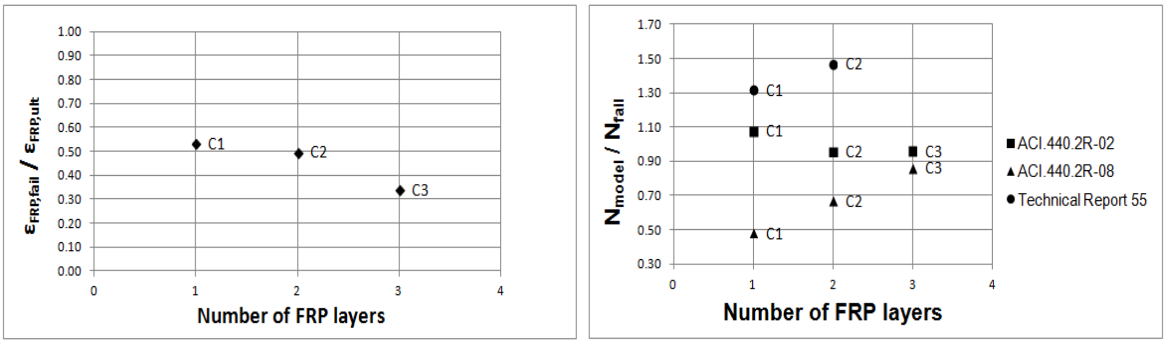


(a) **(b)**

Fig. 4. The design formulae overestimate the failure load of large confined cylinders
(a) $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ ratio **(b)** N_{model} / N_{fail} ratio

Multiple layers of FRPs

When a thick FRP jacket, consists of multiple layers of FRPs is used, complex stress distributions may be developed within the jacket. Furthermore, due to the relatively high stiffness of thick jackets, longitudinal strain may be transferred to the FRP wrap causing significantly high and complex stress field within the FRP laminate. Subsequently, the jacket may fail at a relatively low hoop strain than that of a single-layer jacket. The design codes do not take account of this reduced failure hoop strain of thick FRP jackets. **Fig. 5a** and **Fig. 5b** show the ratios $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ and N_{model} / N_{fail} respectively for three cylinders of 150 mm by 300 mm strengthened with 1, 2 and 3 layers of CFRP respectively (Rousakis and Tepfers [21]) (Table 1). It can be seen from **Fig. 5a** that $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ strain ratio decreases to ~0.35 from ~0.53 when 3 layers are used. Subsequently, this reduces N_{fail} of cylinders with thick FRPs; thus, the empirical models overestimate design failure load (**Fig. 5b**).



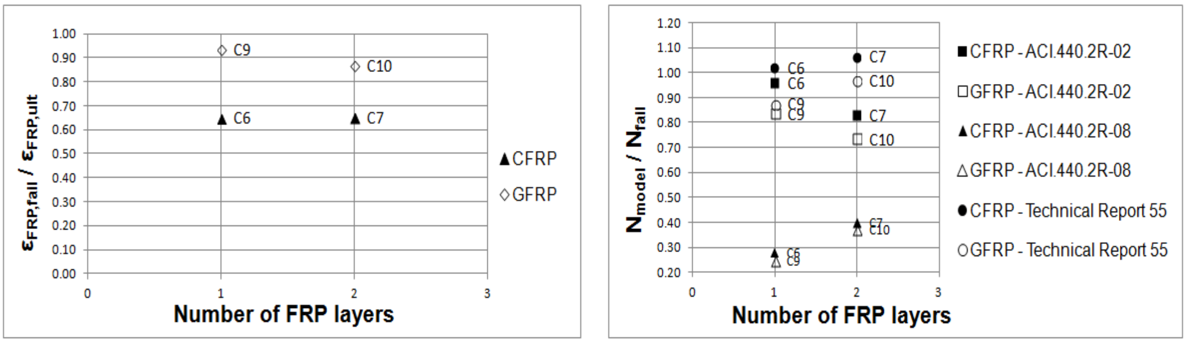
(a) **(b)**

Fig. 5. The FRP rupture strain decreases with the number of FRP layers in the jacket increases
(a) $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ ratio **(b)** N_{model} / N_{fail} ratio

CFRP or GFRP

One of the decisions has to make during the designing of a strengthening application is whether to choose CFRP which has elastic modulus of ~150–250 GPa or to choose low modulus GFRP which has elastic modulus of ~25–100 GPa. Although CFRPs are significantly expensive than GFRPs, most laboratory and real-life strengthening applications reported in the literature were carried out using CFRP because of its high modulus. Although high stiff FRP jackets could provide large confinement forces, the test results show that CFRPs are only strained to ~60% of the ultimate tensile strain. On the other hand, GFRP jackets may be used efficiently exploiting the high strain capacity to achieve similar strength enhancements as that provided by equivalent CFRP jackets. Nevertheless, It should be noted that, in practice, the poor creep properties of GFRPs must be taken into account in designs.

Fig. 6a and **Fig. 6b** show the ratios $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ and N_{model} / N_{fail} respectively for 4 cylinders of 150 mm x 300 mm (chosen from the study of Lam and Teng [16]) strengthened with 1 and 2 layers of CFRP and GFRP respectively (Table 1). It can be seen from **Fig. 6a** that the cylinders strengthened with CFRP, the $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ ratio is ~0.65 whereas that in the GFRP strengthened specimens are ~0.93 and ~0. 87 respectively. Despite its low modulus it can be seen from **Fig. 6b** that, GFRP provides similar strength enhancements as that provided by equivalent CFRP jackets. For instance, the load ratio N_{model} / N_{fail} for the cylinder with 1 layer of CFRP is 1.02 and 0.96 (based on the Concrete Society and ACI 440.2R-02 models respectively), whereas the corresponding strength ratio in GFRP strengthened column is 0.87 and 0.84 respectively. For the cylinder reinforced with 2 layers of CFRP is 1.07 (Concrete Society) and 0.83 (ACI 440.2R-02) whereas that in the equivalent GFRP strengthened cylinder is 0.97 and 0.73 respectively. Thus, the results indicate that GFRP can be used to achieve similar strength enhancements as that provided by equivalent CFRP jackets despite its low modulus.



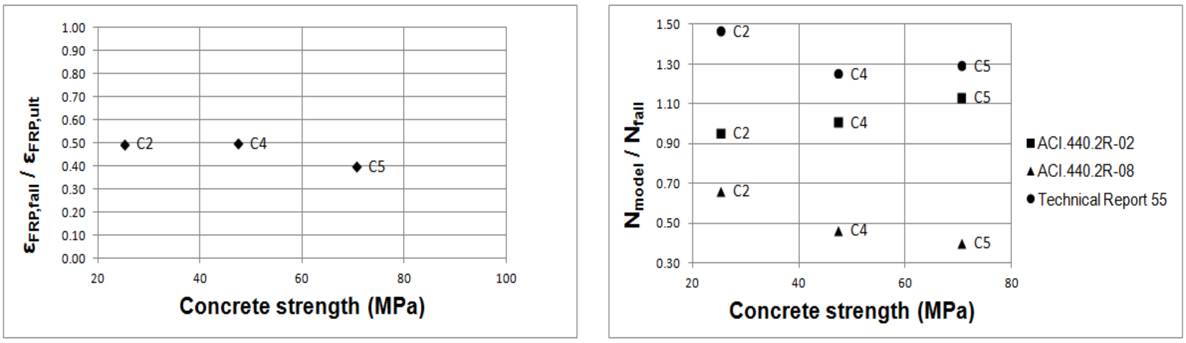
(a) **(b)**

Fig. 6. GFRP jackets fail at a high strain ratio whilst providing same level of confined strength as that provided by CFRP jackets
(a) $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ ratio **(b)** N_{model} / N_{fail} ratio

Effect of concrete strength

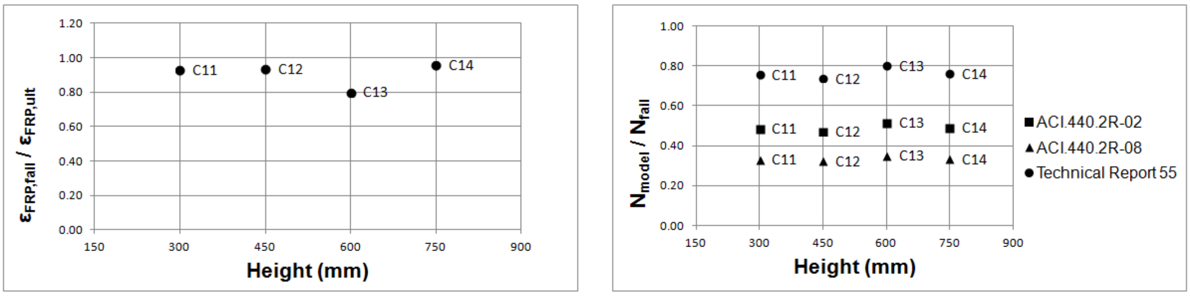
The confined strength of a FRP-confined column primarily depends on the lateral strain developed under applied axial strain, which in turn depends on many factors of the concrete used such as axial shortening, Poisson's ratio's effects, section dilation, cracking and strain limitations etc. It is anticipated that effectiveness of FRP confinement is more significant in low strength concrete than that in high strength concrete. This is due to the high lateral strains and significant dilation (cracks, etc.) develop in low strength concretes at relatively low applied axial strains than that develops in high strength concrete. As an example, the strength enhancement ratio (i.e. ultimate confined axial strength / unconfined strength) of three test specimens cast with concretes of strength 25, 47 and 71 MPa were calculated to be as 2.30, 1.75 and 1.35 respectively, showing a significant influence of the strength of the unconfined concrete on the effectiveness of FRP confinement (The three cylinders, each 150 mm x 300 mm, strengthened with identical 2 layers of CFRP were chosen from the study of Rousakis and Tepfers [21] (Table 1)).

Fig. 7a and **Fig. 7b** show the ratios $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ and N_{model} / N_{fail} respectively for above three specimens. From **Fig. 7a** it can be seen that, in the specimen with high strength concrete the FRP fails at a relatively low strain. From **Fig. 7b** it can be seen that ACI 440.2R-02 code provides better predictions, with N_{model} / N_{fail} ratio close to 1, for all specimens; although an increase from ~0.9 to 1.1 can be seen when the strength of the concrete increases from 25 to 71 MPa. (The Concrete Society model overestimates the strength of the cylinders by at least about 30% and it reiterates the previously discussed observation that the predictions from this model are generally inaccurate for specimens which were not included in the database used in the derivation of the original empirical formula.). The results demonstrate that the code formulae usually overestimate the design load of specimens cast with high grade concretes since the models do not take account of the relatively low lateral strains developed here in comparison to that in an equivalent low strength concrete cylinder.



(a) **(b)**

Fig. 7. The design formulae overestimate the failure load of confined cylinders cast with high strength concrete
(a) $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ ratio **(b)** N_{model} / N_{fail} ratio



(a) **(b)**

Fig. 8. Within the column height / diameter ratio of 2 –5, the height of the column has no effect on the ultimate strength
(a) $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ ratio **(b)** N_{model} / N_{fail} ratio

Effect of column height

A few researches have been carried out in elsewhere [22] to study the effect of column height on the effectiveness of FRP confinement. However, heights of strengthened columns tested in the experimental studies reported in the literature were still chosen to be with a height / diameter ratio not more than ~5: thus, the specimens were still "short columns". In such circumstances, the effect of the FRP confinement will be very similar in all specimens, irrespective of the actual column height. For instance, it was determined that the strength enhancement ratio (i.e. ultimate confined axial strength / unconfined strength) are very similar in four test columns with height / diameter ratios of 2, 3, 4 and 5 respectively, chosen from the study of Silva and Rodrigues [22] (Table 1). **Fig. 8a** and **Fig. 8b** show the ratios $\varepsilon_{FRP,fail}$ / $\varepsilon_{FRP,ult}$ and N_{model} / N_{fail} respectively for above four cylinders with diameter 150 mm and heights 300, 450, 600 and 750 mm respectively, and cast from the same concrete and strengthened with identical FRP jackets. From **Fig. 8a**, it can be seen that all specimens fail at a similar FRP strain, and also the design load predictions from the design code formulae for all columns are very similar (**Fig. 8b**). This results indicate the need for test columns over a larger range of columns heights to study the effect of FRP confinement in real-life columns where columns usually have a higher (typically, >10) height / diameter ratio.

Design FRP strain

Since the design codes are based on FRP rupture failure, which is the most common mode of failure for FRP-confined concrete, the design value of FRP hoop strain must be known in the analysis.

Generally, the ultimate tensile strain of FRP materials are determined from standard flat coupon tests [23]; however, the experimental results of FRP tubes fail under internal pressure show that ultimate hoop strain of the material is significantly lower than that determined from flat coupon tests. Similarly, experimental results show that FRP tensile strength was not researched at the rupture of FRP in FRP-confined cylinders [20]. Despite it is difficult to relate the failure strain of a FRP jacket (ε_{fe}) to the ultimate strain determined from standard coupon tests of the material (ε_{fu}), the design codes generally relate ε_{fe} as a fraction of ε_{fu}. For instance, the design formulae in Concrete Society code and ACI 440.2R-08 recommend using 0.6 times ε_{fu} (both these formulae were originally developed by the same group of researchers). This approximate relationship was derived from a previous work of the same researchers [20], involving 78 test specimens. Despite this approximate ratio 0.6 was only noted for most of the CFRP jackets the corresponding ratio of GFRP and high modulus CFRP jackets considered in the study were 0.85 and 0.79 respectively. The corresponding ε_{fe} / ε_{fu} ratio of the test specimens considered in the current study also show that despite 0.6 can be a reasonable approximate for some CFRP jackets, there is a high ratio (> 0.8) in GFRP jackets (**Fig. 6a**). Furthermore, a significantly low ratio was noted in jackets with multiple layers of FRPs (**Fig. 5a**) and in cylinders of high strength concrete (**Fig. 7a**). The previous version of the ACI code (ACI 440.2R-02) recommends using the lowest of either 0.4% or 0.75 times ε_{fu} as design failure strain of FRP (ε_{fu}), however the code is not providing background information for this recommendation.

In the current study, the predicted design loads for the test specimens considered in the study were recalculated by substituting respective actual failure strain (i.e. observed failure strain in the tests) in the design codes. However, this did not improve the predictions from the models and in fact the comparison between the new model predictions and the observed failure load show an even larger discrepancy and scatter. The authors believe that this is due to the fact that the fitting parameters used in the empirical formulae were derived to match with its own recommend ε_{fe} values, and hence the use of a new value of ε_{fe}, albeit it is the actual failure strain of the given FRP jacket, in fact causes an additional inaccuracy in the empirical models.

CONCLUSIONS

The study has shown that the empirical formulae-based design guidelines recommended by *The Concrete Society Technical Report No. 55*; and *ACI 440.2R-02* and *ACI 440.2R-08* often fail to provide accurate and consistent results for failure load of FRP-strengthened concrete cylinders. It has been shown that although the predictions from *The Concrete Society* model generally agree well with the test data used to derive the empirical formula, it does not provide accurate predictions for the failure load of new test specimens. ACI *440.2R-02* model typically provides better estimates for the confined strength; however, for some test specimens the model underestimates the strength by up to 50%. The study shows that ACI *440.2R-08* model is conservative, and usually underestimates the strength by more than 50%.

The results show that the design codes recommendation for design rupture strain of FRP jackets (usually 0.6 times the ultimate strain determined from flat coupon tests) is not always true, in particular in the case of GFRPs and jackets with multiple layers of FRPs. The study has also shown that substituting respective actual FRP failure strain (i.e. observed failure strain in the tests) in the design codes did not improve the predictions from the models, and it results in an even larger inaccuracy.

There is no independent guidance on the accuracy of the models incorporated in the design codes. The comparisons with test data indicate that the models do not take account of potential differences in the spatial distribution of lateral strain developed in columns of different sizes. Similarly, less effective confinement effect in high strength concrete, observed reduced rupture strain of thick (stiff) FRP jackets, etc. cannot be accounted in the models.

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