Technical University of Civil Engineering of Bucharest

Doctoral School

Application of FRCM for Seismic Rehabilitation of Structures. State of the Art

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Abstract

Reinforced concrete column assumed in existing RC building designed by old codes considered a weak member. The existing RC column analysed by ACI 318-14 [1] and strengthening by fabric reinforced cementitious matrix ‘FRCM’ composite material depending on ACI 549.4R-13 [2]. The shear, axial and flexural strength of the RC column is improved. Analysis and results are presented in chapters one until four. The aims of the report are to determine the percentage of increase in the strength of RC column after strengthened by FRCM. Chapter five present the literature review of previous experimental work in this field. Chapter six present the conclusion. Finally, present the references adopted in the report.

Introduction

The old reinforced concrete buildings designed with poorly detailed reinforcement depending on the old codes without considering seismic design especially for the buildings constructed in the seismic zones are needing to strengthen the members of these buildings (columns in this report) to increase the strength capacity and ductility and enhancement the energy dissipation.

Among the composite materials used for strengthening RC members is fabric reinforced cementitious matrix ‘FRCM’. It can be considered a best composite material to strengthen the RC members, because it has some special features such as, less affected by temperature fluctuations, FRCM is inherently incombustible, possesses porous properties, FRCM can be applied to concrete structural members in low temperature conditions and on wet surfaces, providing a substantial gain in strength and deformation capacity of RC member.

In this report RC column designed as assumed in existing old RC building designed depending on old codes and presented the complete calculation of shear, axial and flexural strength for this existing column before strengthening depending on ACI 318-14[1]. ACI 549.4R-13 [2] is used as a guide to strengthen the RC column with FRCM. Calculation of new shear, axial and flexural strength is completed and finally as mentioned in conclusion part calculate the percent of increase in the strength of existing RC column with FRCM.

The expected behaviour of this RC column is semi-ductile with middle-length and will show flexural- shear failure. This mean the failure starts by the yielding of longitudinal reinforcement within the zones with maximum bending moment and it completed by shear inclined crack. If the premature failure due to shear is prevented, the column can develop substantial (flexural) deformations. [3]

1. Shear Strength Evaluation of Assumed Existing RC Column

In this part of the report, proposed to design a weak RC column like the existing column in an old building designed depending on old codes without seismic design.

This RC column assumed with a cross section (40X40) cm with clear high (150) cm and embedded into two block (90X50X50) cm at ends as fig.2.

The compressive strength of the concrete will be C16/20.

1.1 Failure Mode

The failure mode is depending on the axial force intensity (n), is assumed equal to 0.5 and the aspect ratio (λ) that's calculated by Eq.1:

$$\lambda = \frac{L}{h} \quad \text{———} \quad (1)$$

L: Clear high = 1500 mm.

h: Depth of the cross section = 400 mm.

$\lambda = \frac{1500}{400}$ = 3.75

depending on the (n and λ) values and as the fig.1:

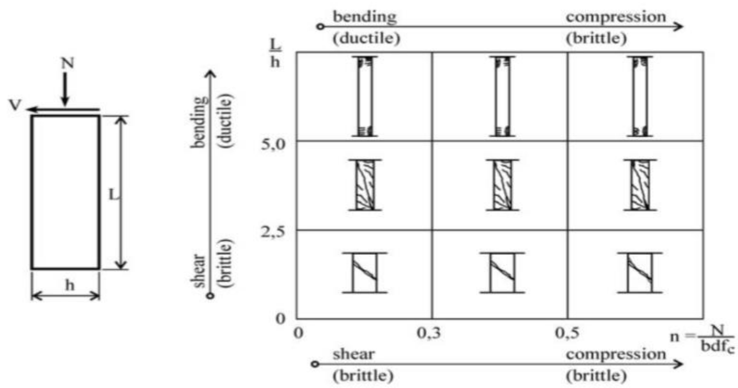

Fig. 1: Types of failure for elements with different aspect ratios and axial force. [3]

The expected behaviour of column is semi-ductile with middle-length and will show flexural-shear failure. [3]

Axial force (N) calculated by Eq.2:

$$n = \frac{N}{b\ d\ f_c} \longrightarrow N = n\ b\ d\ f_c \quad \text{———} \quad (2)$$

Where:

n: Axial force intensity = 0.5.

b, d: Width and depth of the cross section of RC column = (400X400) cm.

f_c: Compressive strength of concert C16/20 = 16 MPA.

N= 0.5x 400x400x16= 1280000 N = 1280 KN

1.2 Steel Reinforcement Evaluation

1.2.1 Longitudinal Reinforcement

Steel reinforcement ratio assumed equal to 0.005 A_g and this ratio is lower than the minimum required ratio as the code, in the code the minimum and maximum reinforcement ratio is 0.01 A_g and 0.08 A_g sequentially as ACI 318-11 [4]. Assumed this ratio is adopted for the purpose to get a weak design for RC column.

Calculation of steel reinforcement by Eq.3:

$$A_s = \rho\ b\ d \qquad (3)$$

Where:

A_s: Area of steel reinforcement.

ρ: refinement ratio = 0.005

b, d: Width and depth of the cross section of RC column = (400X400) cm.

$A_s = 0.005x400x400$ = 800 mm^2

Assume using steel bar with (16) mm diameter, area of bar (As_{bar}) = $\frac{\pi\ D^2}{4}$ = $\frac{\pi\ 16^2}{4}$ = 200.96 mm^2

Number of steel bars reinforcement = $\frac{As}{As_{bar}}$ = $\frac{800}{200.96}$ = 4, then:

Using 4 steel bars for the longitudinal reinforcement.

1.2.2 Shear Reinforcement

Assumed to use Φ 6 @ 200 mm as shear reinforcement bar with yield strength f_{yt} equal to 320 MPA.

Minimum shear reinforcement $A_{v,min}$ shall be greater than (a) and (b):

(a) $0.062\ 0.062\ \sqrt{f_c}\ \frac{b\ s}{f_{yt}}$

(b) $0.35\ \frac{b\ s}{f_{yt}}$

Where:

f_{yt} : Yield strength of steel = 320 MPA.

s: Stirrups spacing = 200 mm.

(a) $0.062\ 0.062\ \sqrt{f_c}\ \frac{b\ s}{f_{yt}}$ = $0.062\ 0.062\ \sqrt{16}\ \frac{400x200}{320}$ = 62 mm^2

(b) $0.35\ \frac{b\ s}{f_{yt}}$ = $0.35\ \frac{400\ x200}{320}$ = 87.5 mm^2

Area of existing shear reinforcement used Φ 6mm, A_v =$\frac{2\ \pi\ D^2}{4}$ = $\frac{2\ x3.14x\ 6^2}{4}$ = 56.57 mm^2

A_v existing $< A_{v,min}$ in (a) and (b), this mean the shear reinforcement is not enough and this good because RC column will be a weak member.

1.2.3 Minimum Hoop Spacing

Depending on the ACI 318-14 [1] the limit ratio of hoop spacing to longitudinal bar steel diameter in potential plastic hinge region to avoid the failure occur by longitudinal steel bar buckling is:

$$\frac{S}{d_b} < 6$$

Where:

S: Hoop spacing = 200 mm

d_b: Diameter of longitudinal steel reinforcement = 16 mm

$$\frac{200}{16} = 12.5 \ > \ 6$$

This mean, the column has not enough hoop spacing in the potential plastic hinge zone.

After designed the steel reinforcement of column and checked the limitations, the details of assumed RC column as fig.2.

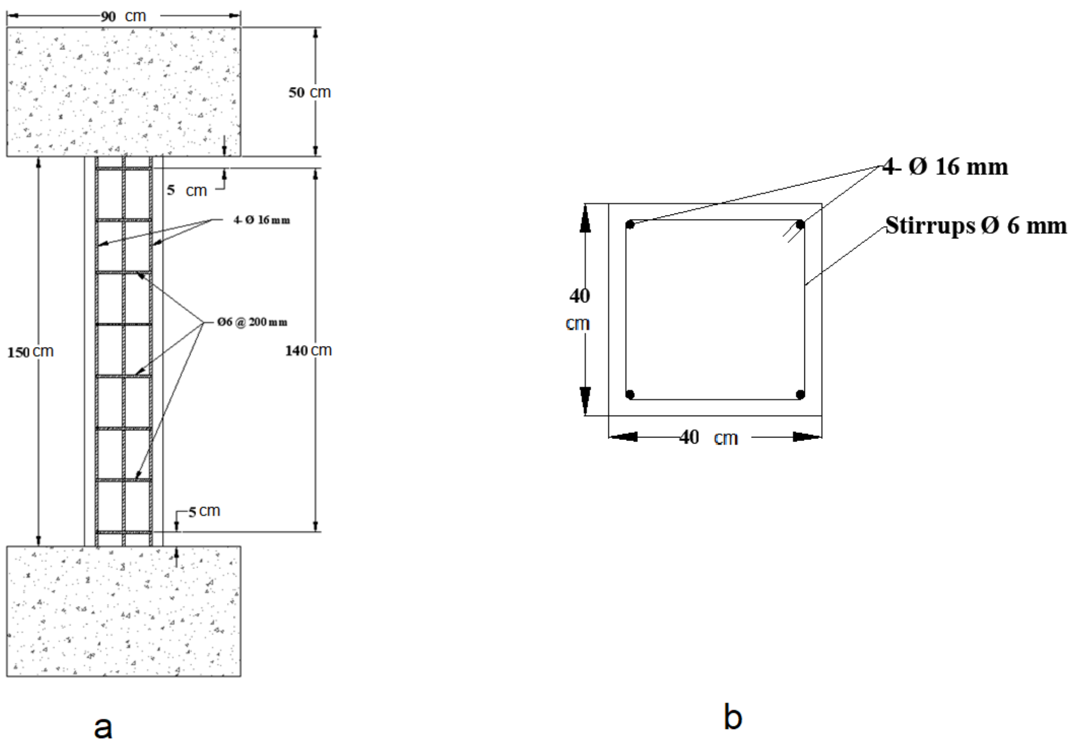

Fig.2: RC column details, a: member geometry. b: section details.

1.3. Calculation of Shear Strength

Depending on the ACI 318-14 [1] code, the nominal share strength V_n calculated by Eq.4:

$$V_n = V_c + V_s \qquad (4)$$

Where:

V_n: Nominal share strength.

V_c: Shear strength provided by concrete.

V_s: Shear strength provided by shear reinforcement steel

As ACI 318-14 code, for nonprestressed member with axial compressing load, V_c calculated by Eq.5:

$$V_c = 0.17\left[1+\frac{N_u}{14\,A_g}\right]\lambda\sqrt{f_c}\;b_w\;d \qquad (5)$$

Where:

N_u: Axial compression load = 1280 KN from eq.2.

λ: Modification factor = 1 according ACI 318-14.

A_g: Cross section area of the column = 400x400 mm.

f_c: Concrete compressive strength = 16 MPA.

$b_w\ d$: width and depth of the cross-section column.

Then,

$$V_c = 0.17\left[1+\frac{1280000}{14\,x400\,x400}\right]x1x\sqrt{16}\,x400\,x400 = 170970 \text{ N} = 170.97 \text{ KN}$$

As ACI 318-14 code, the shear strength provided by shear reinforcement steel V_s is calculated by Eq.6:

$$V_s = \frac{A_v\,f_{yt}\,d}{s} \qquad (6)$$

Where:

A_v: Effective area of shear steel = $\frac{2\,\pi\,D^2}{4} = \frac{2\,x3.14x\,6^2}{4}$ = 56.57 mm^{2}.

f_{yt}: Yield strength of steel = 320 MPA.

d: Depth of the cross-section column.

S: hoop spacing of shear reinforcement = 200 mm.

Then,

$$V_s = \frac{A_v\,f_{yt}\,d}{s} = \frac{56.57\,x\,320\,x400}{200} = 36204.8 \text{ N} = 36.2 \text{ KN}$$

The nominal shear strength V_n is equal:

$$V_n = V_c + V_s = 170.97 + 36.2 = 207.17 \text{ KN}$$

By using the strength-reduction factor Φ is equal to 0.75 from table 21.2.1 in ACI 318-14 [1] code, the nominal shear strength V_n is equal:

ΦV_n = 0.75x207.17= 155.38 KN

2. FRCM Contribution to Shear Strength

The FRCM composite material bonded to surfaces of an existing RC column can be used to enhance the design shear strength by acting as external shear reinforcement.

In this part assumed to use three (3) layers of FRCM with the properties as below:

Area of mesh reinforcement by unit width, A_f =0.046 $\frac{mm^2}{mm}$

Tensile modulus of elasticity, E_f= 124 GPA.

Ultimate tensile strength, f_{fd} =896 MPA.

Ultimate tensile strain, ε_{fd}= 0.0072.

The effective depth of FRCM shear reinforcement d_f is taken equal to effective depth of the steel reinforcement d = 350 mm.

2.1 Calculate the FRCM Design Tensile Strain and Strength

Calculate the FRCM design tensile strain (ε_{fv}) by Eq.7 and design tensile strength (f_{fv}) by Eq.8 are according ACI 549.4R-13 [2].

$$\varepsilon_{fv} = \min(\varepsilon_{fd}\ or\ 0.004) \qquad (7)$$

Then; $\varepsilon_{fv} = 0.004$

$$f_{fv} = \varepsilon_{fv}\, E_f \qquad (8)$$

$$f_{fv} = 0.004\ x\ 124\ GPA\ x1000 = 497\ MPA$$

2.2 Calculate the FRCM Contribution to The Nominal Shear Capacity

The contribution of the FRCM reinforcement to the new nominal shear strength V_f is calculated by Eq.9 according to ACI 549.4R-13 [2]:

$$V_f = 2n\, A_f\, f_{fv}\, d_f \qquad (9)$$

$V_f = 2x3x0.046x497x350 = 48010.2$ N = 48 KN

2.3 Calculate the New Design Shear Strength

The new design shear strength ΦV_n is calculated by Eq.10:

$$\Phi V_n = \Phi\left(V_c + V_s + V_f\right) \qquad (10)$$

$$\Phi V_n = 0.75\ (170{,}97 + 36.2 + 48) = 191.3\ KN$$

2.4 Check Limitation on Total Shear Strength Provided by FRCM and Steel Reinforcement

The limitation indicated in ACI 549.4R-13 [2] should be verified as Eq.11:

$$V_s + V_f \le 8\sqrt{f_c}\, b_w d \qquad (11)$$

36.2+48 = 84.2 KN $<$ $8\sqrt{16}\ 400x400$ = 5120 KN

It's OK

2.5 Check limitation on the shear strength provided by FRCM reinforcement, according to ACI 549.4R-13 [2]. The increment in shear strength provided by the FRCM reinforcement should also not exceed 50 percent of the capacity of the structure without strengthening as Eq.12:

$$V_f \le 0.5\ V_n \qquad (12)$$

48 KN $<$ 0.5x207.17=103.6 KN

It's OK

Finally, after checked all the limitations that adopted in ACI 549.4R-13 [2]. The strengthening steps its right and the result is accepted and the different in values of nominal shear strength are:

The nominal shear strength (ΦV_n) before strengthening with FRCM= 155.38 KN.

The nominal shear strength (ΦV_n) after strengthening with FRCM= 191.3 KN.

The FRCM composite material improved the shear strength of the column about 24%.

3. FRCM Contribution to Axial Strength

For the same details of existing RC column designed in the beginning as fig.2, the axial strengthening is done in this part depending on the ACI 549.4R-13 [2] with FRCM composite material as steps below:

3.1 Calculate the Axial Compressive Strength of Existing RC Column

According to ACI 318R-14 [1] the nominal axial compressive strength (P_n) shall not exceed the maximum axial strength ($P_{n,max}$) :

$P_n \leq P_{n,max}$

Depending on the table 22.4.2.1 in ACI 318R-14 [1] the value of ($P_{n,max}$) is calculated as Eq.13:

$$P_{n,max} = 0.8\, P_0 \qquad (13)$$

$P_0 = 0.85\, f_c\, (A_g - A_{st}) + f_{yt} A_{st}$

A_g: Gross area $= (400x400)$ mm.

A_{st}: Area of longitudinal steel reinforcement $= 800$ mm^2.

$P_0 = 0.85\, x16\, (400x400 - 800) + 320x800$

P_0 = 2421120 N = 2421 KN

Then;

$P_{n,max} = 0.8\, P_0$

$P_{n,max} = 0.8\, x2421 = 1937$ KN

Assume $P_n = P_{n,max}$ = 1937 KN

According to ACI 318R-14 [1] the axial strength reduction factor(Φ) as table 21.2.1 is equal to 0.65, then: The nominal axial compressive strength as Eq. 14:

$$\Phi P_n = 0.65\, P_n \qquad (14)$$

ΦP_n =0.65x1937 = 1259 KN

3.2 Calculate the FRCM Effective Tensile Strain

The FRCM effective tensile strain is computed according to ACI 549.4R-13 [2] as Eq.15:

$$\varepsilon_{fe} = \min(\varepsilon_{fd}\ or\ 0.012) \qquad (15)$$

Where:

ε_{fe}: FRCM effective tensile strain.

ε_{fd}: FRCM ultimate tensile strain, ε_{fd}= 0.0072.

Then:

$\varepsilon_{fe} = \min(0.0072\ or\ 0.012) = 0.0072$

3.3 Select the Number of FRCM Layers

The number of layers of FRCM as assumed in the beginning is equal to three layers, n = 3.

3.4 Calculate the New Axial Strength with FRCM

The efficiency factor (k_a) should be calculated as Eq. 16 when b/h=1:

$$k_a = 1 - \frac{b^2+h^2}{3\,b\,h\,(1-\rho)} \qquad (16)$$

Where:

b, h: Width and depth of the cross section of RC column = (400X400) mm.

ρ: Longitudinal steel reinforcement ratio = 0.005; then:

$$k_a = 1 - \frac{400^2+400^2}{3\,x400x\,400\,(1-0.005)} = 0.329$$

The maximum confinement pressure provided by the FRCM wraps (f_1) should be calculated according to Eq. 17, for rectangular cross-section:

$$f_1 = \frac{2\,n\,A_f\,E_f\varepsilon_{fe}}{\sqrt{h^2+b^2}} \qquad (17)$$

Where:

A_f: Area of mesh reinforcement by unit width, A_f =0.046 $\frac{mm^2}{mm}$.

E_f: Tensile modulus of elasticity, E_f= 124 GPA= 124000 MPA

ε_{fe}: FRCM effective tensile strain = 0.0072.

$$f_1 = \frac{2x3x0.046x124000x0.0072}{\sqrt{400^2+400^2}} = 0.435 \text{ MPA}$$

The maximum concrete confined compressive strength (f_{cc}) should be calculated according to Eq.18. For rectangular cross-section:

$$f_{cc} = f_c + 3.1\,k_a\,f_1 \qquad (18)$$

f_c: Concrete compressive strength

k_a: The efficiency factor = 0.329.

f_1: The maximum confinement pressure provided by the FRCM wraps = 0.435 MPA.

Then;

$f_{cc} = 16 + 3.1x0.329x0.435$

$f_{cc} = 16.44$ MPA

The new design column axial strength is calculated according to ACI 318-11 as Eq.19:

$$\Phi P_{nNew} = \Phi 0.8 \left[0.85\, f_{cc}\, (A_g - A_{st}) + A_{st}\, f_{yt}\right] \qquad (19)$$

$\Phi P_{nNew} = 0.65 \text{ x } 0.8\ [0.85\ x 16.44 x\ (400 x\ 400 - 800) + 800 \text{x } 320]$

$\Phi P_{nNew} = 1290$ KN

3.5 Check Capacity

The new design column axial strength should be greater than original design axial strength as ACI 549.4R-13 [2]:

$\Phi P_{nNew} \geq \Phi P_n$

$\Phi P_{nNew} = 1290 > \Phi P_n = 1259$

It’s OK.

And the increase in axial strength provided by the FRCM reinforcement shall not exceed 20 percent of the existing capacity of the column without strengthening:

$(\Phi P_{nNew} - \Phi P_n) \leq (0.2\ \Phi P_n)$

$(1290 - 1259) \leq (0.2 \text{ x} 1259)$

$31 < 251.8$

It’s OK.

The FRCM composite material improved the axial strength of the column about 3%.

4. FRCM Contribution to Flexural Strength

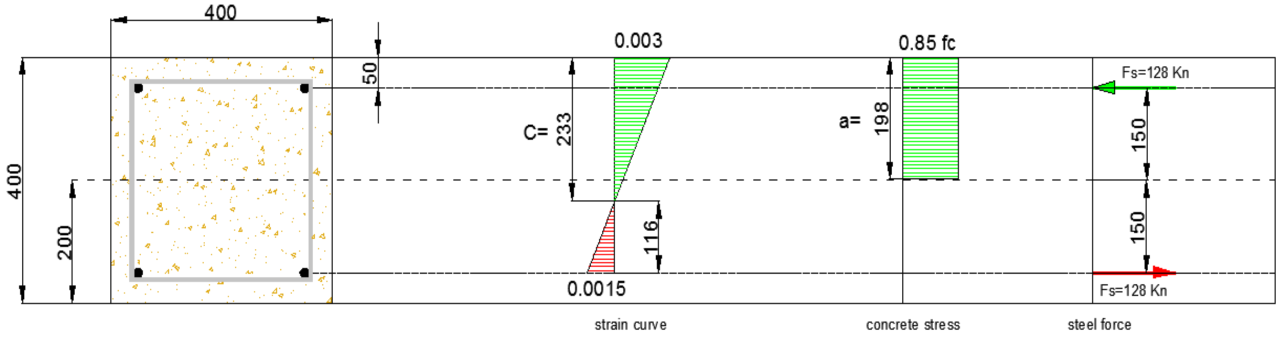

Fig.3: Stress and force analysis, the dimensions in "mm".

4.1 Calculate the Flexural Strength of Existing RC Column:

$$\varepsilon_s = \frac{f_{yt}}{E_s}$$

ε_s : Strain of steel;

f_{yt} : Yield stress of steel;

E_s : Modulus of elasticity of steel.

$$\varepsilon_s = \frac{320\ MPA}{207000\ MPA} = 1.5x10^{-3}$$

Calculate (C), the depth of compression strain in concrete:

By interpolation:

$$\frac{3x10^{-3}}{C} = \frac{1.5x10^{-3}}{350 - C}$$

$1050x10^{-3} – 3x10^{-3}$ C= $1.5x10^{-3}$ C

$1050x10^{-3}$= $1.5x10^{-3}$ C+$3x10^{-3}$ C

1050=4.5C

C=233 mm

Calculate (a), the depth of compression stress of concrete according ACI 318R-14 [2]:

$$a = \beta C$$

As code ACI 318R-14 [1] β=0.85, then:

a= 0.85x233=198 mm

Calculate the tension and compression force of steel bar (F_s):

F_s= $f_y x A_{st}$

F_s= 320x200x2=128 KN for each direction compression and tension.

Calculate the nominal flexural strength (M_n):

Depending on fig.3:

M_n = 128x150x2x10^3 + 0.85x16x198x400x100

M_n=146 KN.M

4.2 Calculate the FRCM Effective Tensile Strain:

Effect tensile strain level in the FRCM reinforcement attained at failure (ε_{fe}) should be limited to the design tensile strain of the FRCM composite material (ε_{fd}). According ACI 549.4R-13 [2] the limit as Eq.20:

$\varepsilon_{fe} = \min(\varepsilon_{fd}\ or\ 0.012)$ ——— (20)

$\varepsilon_{fe} = \min(0.0072\ or\ 0.012)$

$\varepsilon_{fe} = 0.0072$

4.3 The effective tensile stress level in the FRCM reinforcement attained at failure (f_{fe}) in the FRCM reinforcement is calculated in accordance ACI 549.4R-13 [2] as Eq.21:

ε_{fe}=$E_f\varepsilon_{fe}$ ——— (21)

ε_{fe}= 124000x0.0072

ε_{fe}= 892.8 MPA

4.4 Calculate the FRCM force (F_f) as Eq.22:

$F_f = \varepsilon_{fe}\ xA_f x\ n$ ——— (22)

$F_f = 892.8x.046x800x3$

$F_f = 98.5\ KN$

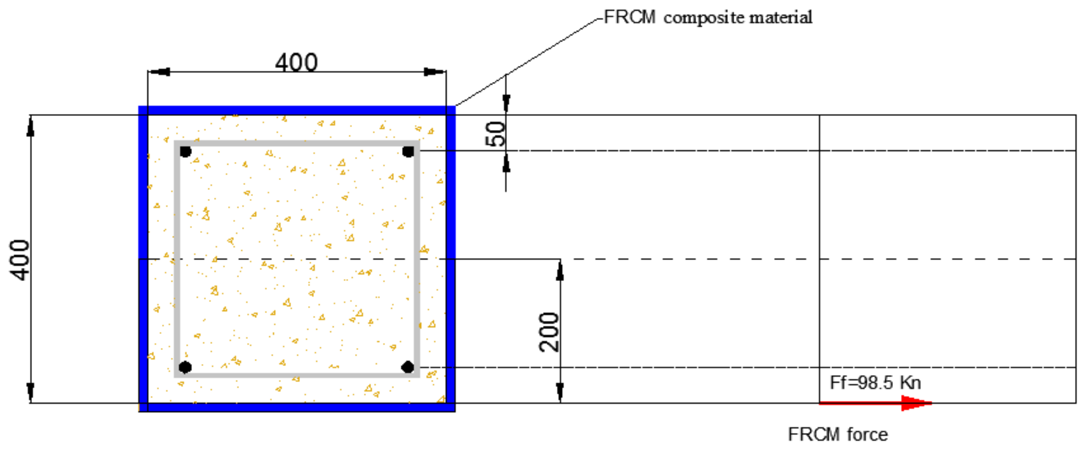

Fig.4: FRCM force in section.

4.5 Calculated the contribution of FRCM composite material to the nominal flexural strength as Eq.23:

$$M_f = F_f x R \qquad (23)$$

R: The arm of moment = 200 mm, as fig.4.

$M_f = 98.5x0.2$

$M_f = 19.7\ KN$.M

4.6 The design flexural strength is calculated as ACI 549.4R-13 [2] by Eq24:

$$\Phi_m M_n = \Phi_m\left[M_s + M_f\right] \qquad (24)$$

Where:

Φ_m: Strength reduction factor = 0.9.

M_n: Total nominal flexural strength;

M_s: Nominal flexural strength before strengthening;

M_f: The contribution FRCM composite material to the nominal flexural strength.

$\Phi_m M_n = 0.9[149 + 19.7]$

$\Phi_m M_n = 151.8$ KN.M

4.7 Design limitation:

The increase in flexural strength provided by the FRCM reinforcement should not exceed 50 percent of the existing flexural capacity of the structure without strengthening ACI 549.4R-13 [2]:

$M_f \leq 0.5\ M_n$

19.7 ≤ 0.5x149

$19.7 < 74.5$

It's OK.

Finally, after checked the limitation that adopted in ACI 549.4R-13 [2]. The strengthening steps its right and the result is accepted and the FRCM composite material improved the flexural strength of the column about 13%.

5. Literature Review of Previous Experimental Study

5.1 Strengthening Column by PBO-FRCM Wrapping System

The experimental study [5] used a reinforced concrete column with rectangular cross sections strengthened by cement based composite materials wrapping system.

Fabric meshes of PBO (short of Polypara-phenylene-benzo-bisthiazole) fibers embedded into a cement-based matrix (PBO-FRCM system). The PBO fabric mesh is formed by roving 10 mm and 20 mm spacing in the two orthogonal directions as fig. 5. The nominal equivalent thickness in the two fibers directions are 0.0455 mm (longitudinal direction) and 0.0224 mm (transversal direction).

After the application of the first mortar layer on the concrete surface with a thickness of 4 mm, the first layer of PBO fabric mesh was applied and slightly pressed into the mortar. The next mortar layer with a thickness of 4 mm covered the PBO fabric mesh completely and the operation was repeated until all the PBO layers were applied and covered by mortar. Strengthened specimens were left to cure.

Fig. 5: PBO fabric mesh. [5]

The specimens were eccentrically loaded in compression and tested until the collapse. The tested specimens were subdivided in two series as fig.6.

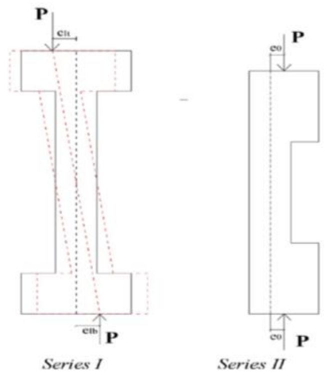

Fig. 6: Load eccentricity of tested specimens. [5]

Each specimen was subjected to axial force combined with bending moment variable along the height; the section at the top of the columns was subjected to bending moment opposite in sign respect to that acting at the bottom section.

The details of the two series as in fig. 7:

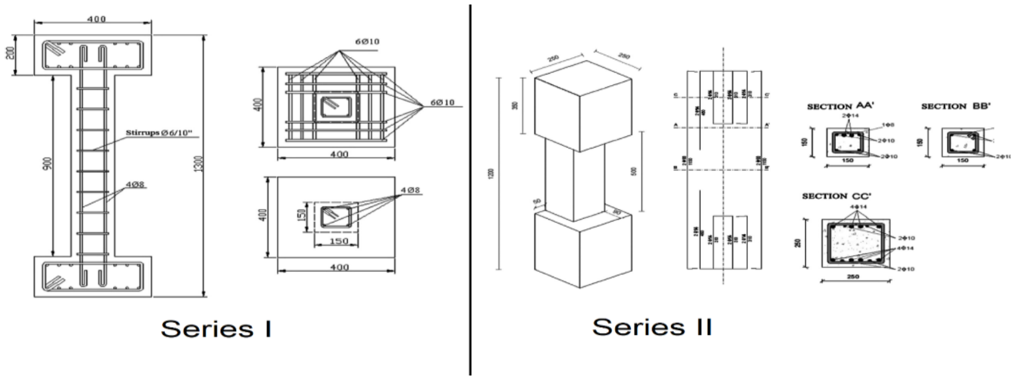

Fig.7: Details of the two series. [5]

Series I:

1.Failure Modes:

Depending on the eccentricity as the two-eccentricity used 24,27 mm:

- For e24 mm the failed at the top of the column by rupture of the PBO fabric mesh and crushing of the concrete. While, for e27 mm the failure occurred at the bottom of the column through yielding of steel bars followed by the concrete crushing without rupture of the PBO fabric meshes;
- For e24mm at failure, considerable lateral displacements were recorded at the top of the column where significant cracks both longitudinal and transversal were observed. While, for e27 mm lateral displacements were recorded at failure at the top of the column but no crack formation was observed in the column;
- The ultimate strength of the confined columns decreases while increasing the eccentricity value (from 24 to 27 mm) for both specimens were confined with the same amount of fibres. So, that the failure of the confined column occurred at 486.5 KN and 456.12 KN for the columns with eccentricity 24,27 mm respectively.

2.Load-Strain Diagrams:

A- Axial strain as fig.8:

- The behaviour of all specimens was liner until the peak load;
- The diagram relative to the specimens with eccentricity 27 mm has a slop higher than of the diagram of specimens with e=24;
- The behaviour of specimens with e=27 is more brittle.

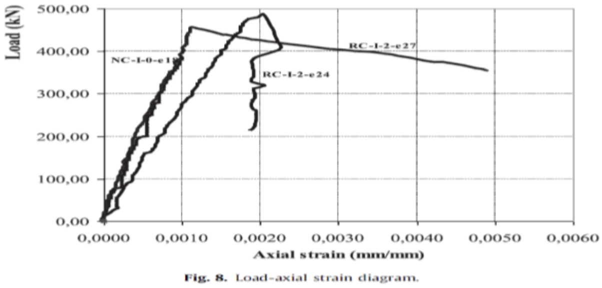

Fig. 8: Load-axial strain diagram. [5]

B- lateral strain as fig. 9:

- When the lateral strains variation recorded at the top of the specimens as fig. 9 a. The strain value of the specimen with e=27 is higher than of the specimens and when the lateral strain variation recorded at the bottom as fig.9 b, then the specimen with e=24 be higher.

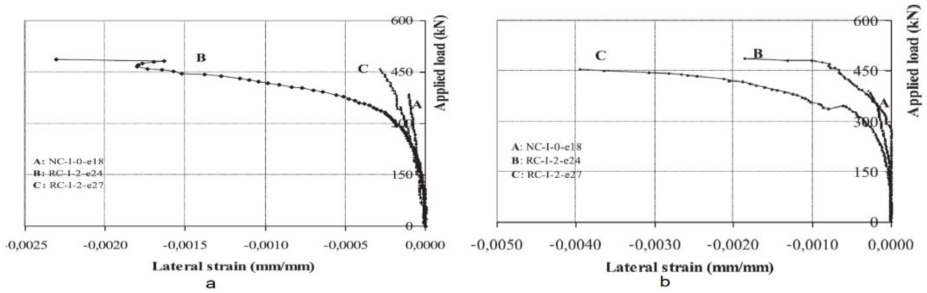

Fig. 9: Load-lateral strain diagram at: a: the top of specimens, b: at the bottom of specimens. [5]

Series II

1. Failure Mode:

Depending on the eccentricity as the two-eccentricity used 50,30 mm:

The failure mode dependent on the confinement ratio while the eccentricity values are not influential (50 and 30) mm. Because the failure mode of the specimens confined with single layer with eccentricity 50 and 30 was due to fibre break, crushing of the concrete and buckling of the compressed internal steel bars. While double layers confined of specimens with e=50,30 failed by crushing of the concrete without rupture of fibres.

About the failure load, its increased when the eccentricity decrease with the same number of PBO-FRCM layers, for example; the specimens with one layer and e=50 and 30, the failure load increase of 3.17 and 14.51, respectively. Finally, the strength of the confined column is depending both on the eccentricity values and the confinement ratio.

2. Lateral Deflection of Confined Specimens:

In fig.10 the curve of lateral deflection reported by using values measured by horizontal LVDTs positioned at 400 mm(bottom), 600 mm(mid-high) and 800mm(top) from the base of specimens.

The lateral displacements of double layers confined specimens are higher than those of single layer confined specimens.

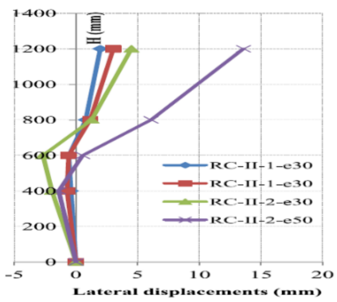

Fig. 10: Lateral deflection of confined specimens. [5]

5.2 Confined Columns with CFRCM Jacket Under Cyclic Load

The experimental study [6] for the confined columns with carob reinforced cementitious matrix 'CFRCM' jacket under cyclic load. Used six specimens with three types of cross sections (circular, square and rectangular) with constant height equal 60 cm.

The carbon mesh used with 10X10mm, XMESH10. Two orthogonal directions with mortar binder 3 mm thickness.

Application of the CFRCM jacket required moistening in water and application of the binding mortar in a layer about 3 mm thick and apply the carbon fiber mesh in it. Afterwards it was necessary to apply a second layer of binding mortar about 3 mm thick to cover the mesh completely.

When several layers of fiber mesh were required, the operation described above was repeated, wet on wet, by applying the next layer when the previous layer was not completely hardened. At the end of the fiber mesh a 100 mm overlap was added.

The confinement columns with CFRCM jackets can enhance the performance of unconfined columns with a substantial gain in strength, ductility and energy absorption capacity.

1. Effectiveness of CFRCM Jacket

CFRCM jackets did not fail suddenly because in the case of inorganic matrix composite jackets the fracture initiates from a limited number of fiber bundles and then propagates rather slowly in the neighbouring bundles, resulting in a more ductile failure mechanism. This phenomenon is since the fiber mesh in the inorganic matrix is usually not well impregnated and loading of the fibers is done with non-uniform distribution of forces, leading to 'telescopic failure'. The term "telescopic" is used here to describe the relative slip between fibers in the outer part of each roving and those in the core. Furthermore, multiple cracking in the matrix of CFRCM jackets at low tensile stress levels leads to pull-out failure at higher stress levels, rather than sudden fracture of the fibers.

In all cases the failure mechanism was due to jacket collapse at the corner as fig. 11.

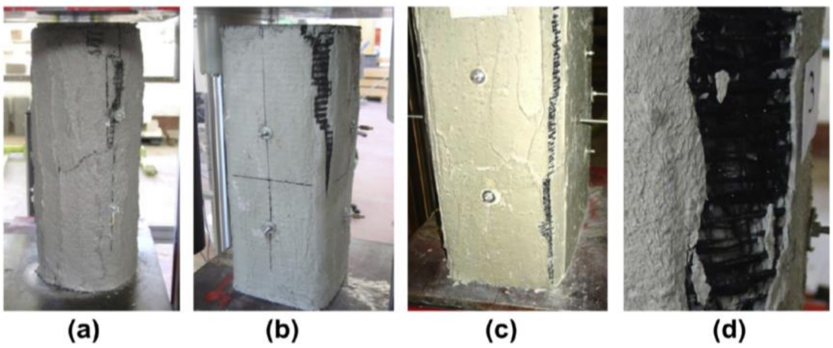

Fig. 11: Failure mode for columns with cross section: (a) circular; (b) square; (c) rectangular. (d) Tensile fracture of the carbon fiber mesh at the corner. [6]

2. Effect of The Number of CFRCM Layers

1- It should be point out that the strength increases in series circular, square and rectangular cross sections having the two fiber layers is roughly the same, as fig. 12;

2- To get the same strength level for the circular section with three FRCM layers, should use four layers in the square and rectangular sections, this is because the corners of these sections lead to decrease the behaviour efficiency of FRCM, as fig.12;
3- For circular columns the increases in absorbed energy is negligible, while for square and rectangular columns the ultimate absorbed energy increases 70% and 20%, when four instead of two layers of CFRCM respectively are used as fig. 13.

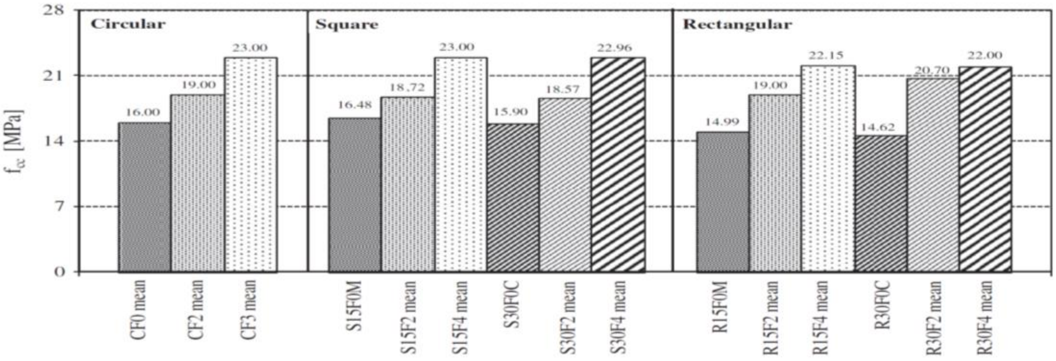

Fig.12: Effectiveness of CFRCM jacket on the peak stress. [6]

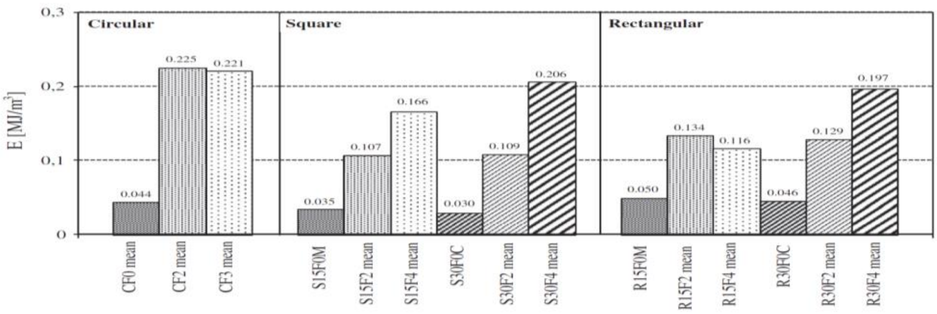

Fig. 13: Effectiveness of CFRCM jacket on the energy absorption capacity. [6]

3. Effect of Cyclic Loading on Stress Deterioration

In order to evaluate the degree of stress deterioration for each cycle a stress deterioration ratio β_1 was defined as follows:

$$\beta_1 = \frac{f_{new}}{f_{un.env}}$$

Where referring to the symbols in fig. 14, $\varepsilon_{un.env}$ and $f_{un.env}$are the axial strain and the corresponding axial stress at the unloading point (point A), $\varepsilon_{re}$and f_{re} the axial strain and corresponding stress at the beginning of the reloading (point b), f_{new} is the stress in the reloading branch at the deformation value $\varepsilon_{un.env}$ (point c) where the unloading started, $\varepsilon_{ret.env}$ and $f_{ret.env}$the strain and corresponding stress at the return on the envelope curve (point d).

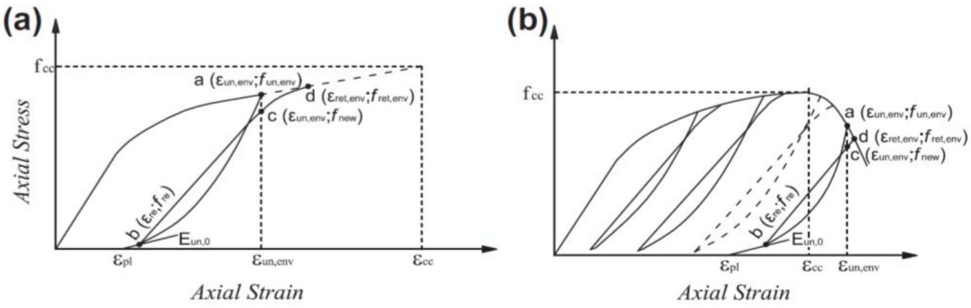

Fig. 14: Key parameters for evaluation of stress deterioration and stiffness loss under unloading/reloading cycles: (a) hardening branch; and (b) softening branch. [6]

Fig. 15a and b shows the correlation between the strain $\varepsilon_{un.env}$ where the unloading path starts and β_1, for specimens with medium and high lateral confinement pressure respectively. These figures show that for small envelope unloading strains ($\varepsilon_{un.env} < 0.001$), β_i is almost equal to 1, and the strength degradation is negligible. As $\varepsilon_{max,i}$ increase, β_i tends to decrease and reaches a constant value of about 0.93 for both specimens with medium and high lateral confinement pressure.

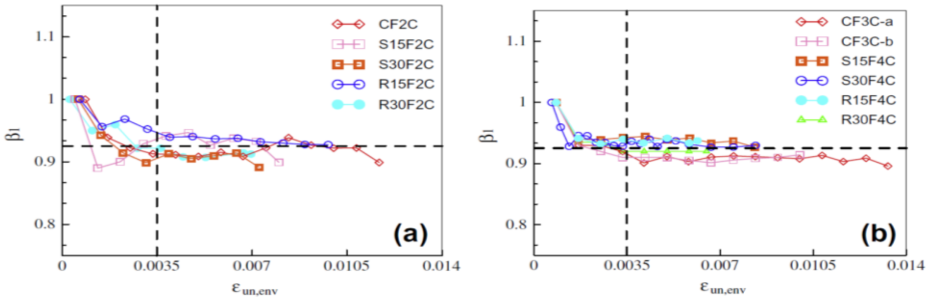

Fig. 15: Stress deterioration under unloading/reloading cycles: (a) columns with medium lateral confinement pressure; and (b) columns with high lateral confinement pressure. [6]

4. Failure Mode and Crack Patterns

In fig. 16 a comparison is shown between the stress–strain curve, normalized with respect to the peak value, and the corresponding decay in the normalized secant stiffness–strain curves for some specimens used to describe and discuss the main damage patterns and the failure mechanisms for CFRCM confined columns.

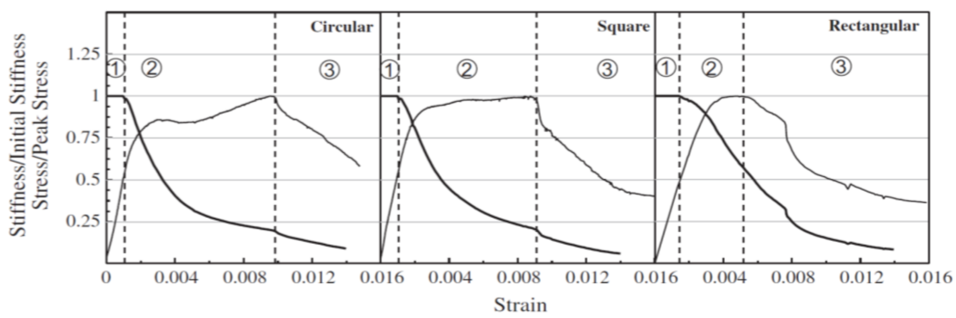

Fig. 16: Stress–strain and stiffness–strain curves for circular, square and rectangular columns. [6]

As fig. 16 three stages are appeared:

1. Stage one, the stress–strain curves are characterized by a linear ascending branch and the secant stiffness– strain curves are characterized by a constant branch;
2. Stage two, characterized by nonlinear ascending stress–strain curves and stiffness decreases. In this region the cracked concrete core fully interacted with the elastic CFRCM jacket with a redistribution of internal forces across the mortar. At peak load, due to expansion of the concrete core, multiple sub-vertical cracking in the CFRCM jacket occurred;
3. Stage three, beginning at the point where the slope of the stress– strain curves and the stiffness–strain curves dropped suddenly, the failure mechanism was activated.

6. Conclusion

Strengthening of existing RC column by FRCM composite material to increasing the strength (shear, axial and flexural) of this member is good technique depending on the results that obtained. As described in this report by numerical calculation, the FRCM composite material increased the shear strength of RC column about 24 % its good result especially in seismic area, because the brittle shear failure is important thing take it into account. The axial strength increased about 3 % and flexural strength about 13 %. This mean the load ultimate capacity of existing RC column will increase and the failure will be occurred at late time compared with non- strengthening member. In same time the section geometry of this member did not change a lot because FRCM composite material is thin material.

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