Finite element modeling of reinforced concrete columns seismically strengthened through partial FRP jacketing

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SUMMARY:

The present analytical study concerns the finite element modeling of existing columns of low concrete strength externally strengthened through partial glass FRP sheet confinement. It uses the results of an extensive experimental program that concerns plain and reinforced concrete square columns with side 150 mm and 750 mm height having low concrete strength and internal steel reinforcement of different quality (smooth S220 or ribbed B500C steel bars). Twelve specimens were strengthened by glass FRP sheet confinement in four different configurations: full wrapping, one strip of 50 or 65 mm width or 2 layers of 40 mm width (partial wrapping). The finite element models use a Drucker-Prager type plasticity model for concrete. The analytical predictions compare well with the experimental stress-strain curves. The variation of stress in concrete and in FRP wraps is presented for partial or full confinement. The effects of different quality bars are also discussed.

Keywords: Finite element analysis, concrete columns, concrete plasticity, confinement, FRP straps

1. INTRODUCTION

A large number of existing reinforced concrete columns may present inadequate reinforcement and detailing when compared with the current codes' requirements. Thus, they may perform poorly when subjected to design seismic actions (or higher). The retrofit of such columns includes external confinement on concrete to meet current codes' concrete strength and deformability demands. Numerous experimental and analytical studies look into the full Fiber Reinforced Polymer (FRP) sheet wrapping of columns (Rochette and Labossiére 2000; Matthys 2000; Fam and Rizkalla 2001; Karabinis and Rousakis 2002, 2003; Campione and Miraglia 2003; Lam and Teng 2004; Teng and Lam 2004; Rousakis 2005; Silva and Rodrigues 2006; Lam et al. 2006;; Al-Salloum 2007, Rousakis & Karabinis 2008 and 2011, among others).

Partial wrapping (in between existing stirrups) has been given little attention despite that it may be an efficient and economic design approach, considering that a mechanical volumetric ratio of transverse reinforcement already exists in the columns. The study by Barros and Ferreira (2008) has involved circular plain or reinforced concrete columns with different concrete strengths and with a variety of carbon FRP straps configurations.

The present analytical study concerns the strengthening of existing columns of low concrete strength through partial glass FRP sheet confinement. It uses the results of an experimental program at Reinforced Concrete Laboratory in Democritus University of Thrace that concerns reinforced concrete columns of low concrete strength with square section. The columns vary by the quality of the steel bars and by the configurations of the partial glass FRP sheet wrapping. The paper focuses on the results of the 3-dimensional finite element analyses of the columns using an advanced Drucker-Prager type plasticity model proposed for FRP confined concrete (Rousakis 2005, Karabinis et al 2008). The analyses are performed with ABAQUS software.

2. FINITE ELEMENT MODELLING

2.1. Drucker – Prager type plasticity model for FRP confined concrete

The FE analytical models of the columns are constructed in ABAQUS finite element software. Concrete modeling uses a Drucker–Prager type material. Drucker – Prager type concrete material modeling has been proved very efficient for steel confinement applications (Karabinis and Kiousis 1996 among else). The concrete material model was further developed for FRP confinement applications. The required material parameters used in this study are based on the research by Rousakis (2005), Rousakis et al. (2007), Karabinis et al. (2008).

Concrete is modeled using a solid eight-node element. The elastic response of concrete is considered linear following Hooke's law:

 $\{\sigma\} = [E]\{\varepsilon^e\}$ (2.1)

where $\{\sigma\}$ is the strain tensor, [E] is the constitutive matrix of elasticity, $\{\epsilon\}$ is the strain tensor and the superscript "e" indicates elastic strain. Nonlinear concrete modeling follows a Drucker – Prager failure criterion :

$$F = \sqrt{J_{2D}} f(K) + \theta J 1 - k = 0 \tag{2.2}$$

where J_{2D} is the second invariant of stress deviator, J_1 is the first invariant of stress and θ is the friction parameter. Function f(K) is an indirect expression of Lode's angle combining the third and second invariant of the deviatoric stress (symbolized as J_{3D}). It accounts for the variation of the shear strength of concrete for different load paths at a given hydrostatic pressure and determines the shape of the failure function in the deviatoric plane.

The hardening-softening parameter κ is derived by the relation:

$$\kappa = (1/\sqrt{3} - \theta)\sigma_c \tag{2.3}$$

where σ_c is the unconfined concrete strength.

The required plastic potential function G is of Drucker – Prager type:

$$G = \sqrt{J_{2D}} f(K) + f(\alpha) J_1 \tag{2.4}$$

where $f(\alpha)$ is an expression of the parameter of plastic dilatation of concrete that affects the direction of plastic strain vector. A non-associated flow rule is considered.

The angle of friction and the dilation angle used in the finite element analyses for the FRP straps is a further development of the Karabinis et al. (2008) analyses of full FRP confinement for use in FRP straps. In the present work the parameter of friction required for the plasticity model depends on the concrete strength while the plastic dilation is calculated through expressions related to the plain concrete strength and to the confinement modulus for non uniform confinement. The values used in the present work are 40.48° for the friction angle and 36.88° for the dilation angle which corresponds to dilation parameter a=1.5.

A solid eight-node element is used for the steel reinforcement. Its hardening behavior is modeled. FRP jacket is modeled as quadrilateral lamina element with membrane properties (M3D4R), linear reduced Gauss integration points (one point per element), and enhanced hourglass control. FRP response is

considered orthotropic linearly elastic up to failure. Displacement compatibility is considered between the concrete and the steel or the composite material in the axial or lateral direction.

2.2. Mode of loading and meshing

Monotonic axial loading is imposed concentrically on a rigid plate that in turn loads uniformly exclusively the concrete surface and not the FRP jacket. A quarter of the column is modeled, utilizing symmetry in loading and geometry. The meshing aims at simplicity and compatibility of nodes position among elements in contact.

3. FINITE ELEMENT ANALYSIS RESULTS AND DISCUSSION

An experimental investigation of reinforced concrete columns externally confined by FRP sheets or reinforced concrete jacketing subjected to axial compressive load has been performed in the Laboratory of Reinforced Concrete of Democritus University of Thrace. It concerns plain and reinforced concrete square columns with side 150mm and 750 mm height with 25 mm corner radius. The columns have low concrete strength (average strength of $f_c=13.4$ MPa) and two types of internal steel reinforcement: smooth S220 or ribbed B500C steel bars (Fig. 3.1). The used FRP jacket is an S&P G-sheet E-glass reinforcement of 73 GPa modulus of elasticity and 0.154 mm thickness, impregnated with S2WV Sintecno resins. Twelve specimens are presented here (Table 3.1) that are confined by the glass FRP sheet in four different configurations: full wrapping (a), one strip of 50 (b) or 65 mm (c) width or 2 layers of 40 mm (d) width (partial wrapping).

Series	Column	Reinforcement			
		Longitudinal	Transverse		
		reinforcement	reinforcement	GFRP	GFRP layers
		(Ø8)	(Ø5.5)		
S0	SG1	nlain concrete		Full wrapping	1 layer
	SG1W65			Partial wrapping	1 layer
	SG1W50	plain concrete		Partial wrapping	1 layer
	SG2W40			Partial wrapping	2 layers
S1	RCS1G1	\$220	S220	Full wrapping	1 layer
	RCS1G1W65			Partial wrapping	11ayer
	RCS1G1W50			Partial wrapping	1 layer
	RCS1G2W40			Partial wrapping	2 layers
S2	RCS2G1	B500C	S220	Full wrapping	1 layer
	RCS2G1W65			Partial wrapping	11ayer
	RCS2G1W50			Partial wrapping	1 layer
	RCS2G2W40			Partial wrapping	2 layers

 Table 3.1 Columns designation



Figure 3.1. Dimensions and steel reinforcement detailing

The stirrups are placed at 100 mm distance at the middle height and closer near the ends of the column

to avoid undesirable boundary failures. The discretization of concrete, steel reinforcement and GFRP is illustrated in Fig. 3.2. The categories of 1 layer of GFRP sheet of 65mm width strap and that of 2 layers of GFRP sheet of 40 mm width strap were designed equivalent, following the approaches of existing recommendations.



Figure 3.2. Discretization of concrete (a), steel reinforcement (b), GFRP full wrapping (c), partial wrapping of 65 mm width GFRP layer (d), partial wrapping of 50 mm width GFRP layer (e), partial wrapping of 40mm width GFRP layer (f)

The analytical curves of stress versus strain are compared with the experimental results for all three series of specimens (plain concrete and reinforced concrete specimens). For the plain concrete columns SG1, SG1W65, SG1W50 and SG2W40 the FE models provide fair accurate analytical stress-axial strain curves close to the experimental ones. Yet, the post-elastic experimental behaviour of SG1W50 and SG2W40 is rather softening.



Figure 3.3. Finite element analysis predictions versus stress – strain experimental results for plain concrete specimens SG1 (a) and SG1W65 (b)



Figure 3.4. Finite element analysis predictions versus stress – strain experimental results for plain concrete specimens SG1W50 (a) and SG2W40 (b)

A divergence of the elastic branch is observed between the analytical and experimental curves of the reinforced concrete specimens. This may be attributed to the difference between the boundary conditions of the real column and the FE model. As shown in Fig. 3.1 the longitudinal bars are anchored inside the concrete core and the load is transferred through concrete to them. On the other hand, in the FE model, the bars are considered directly fixed on the loading rigid plate. The loading plate compresses directly both the concrete section and the steel bars. Thus, the initial modulus of the FE model is higher than the experimental one. The post-elastic analytical stress-strain curves coincide with the experimental results. In specimens RCS1G1W50, RCS2G1W50 and RCS2G1W65 the stresses observed from the finite element analysis are higher than the stresses observed from the stresses observed from the stresses observed from the stresses are the stresses observed from the stresses obse



Figure 3.6. Finite element analysis predictions versus stress – strain experimental results for reinforced concrete specimens RCS1G1 (a) and RCS1G1W65 (b)



Figure 3.7. Finite element analysis predictions versus stress – strain experimental results for reinforced concrete specimens RCS1G1W50 (a) and RCS1G2W40 (b)



Figure 3.8. Finite element analysis predictions versus stress – strain experimental results for reinforced concrete specimens RCS2G1 (a) and RCS2G1W65 (b)



Figure 3.9. Finite element analysis predictions versus stress – strain experimental results for reinforced concrete specimens RCS2G1W50 (a) and RCS2G2W40 (b)

Figures 3.10, 3.11 and 3.12 illustrate the variation of concrete principal confining stresses (contour mode) in plain and reinforced concrete specimens along their whole height at the experimental failure strain value. Also, the region of the slice of the highest confining stress is arrowed. The variation of the compressive stress field (negative stress for compression) is also presented in the section plane (concrete slice) at the region of the higher compressive corner value of each column. The confining stresses are higher in the region at the corner of the section. Those stresses vary significantly across the height of the member for narrower straps or higher thickness of the straps. So, for plain concrete columns (Figure 3.10), the highest corner confining stress is remarked in the FRP strap region of 2 layers of 40 mm strap similar to the one of full confinement. The lowest corner confining stress in the FRP strap region is observed for 65 and 50mm strap (however there is a qualitative divergence between the experimental softening stress-strain behaviour and the predicted hardening behaviour in 50 mm strap as well as in 40 mm strap). The corner confining stresses between the straps are around ten times lower than the ones at the strap region and have around the same magnitude in columns with different strap width.



Figure 3.10. Variation of concrete principal confining stresses in section plane (contour mode) on the plain columns SG1 (a), SG1W65 (b), SG1W50 (c), SG2W40 (d)



Figure 3.11. Variation of concrete principal confining stresses in section plane (contour mode) on the reinforced concrete columns RCS1G1 (a), RCS1G1W65 (b), RCS1G1W50 (c), RCS1G2W40 (d)



Figure 3.12. Variation of concrete principal confining stresses in section plane (contour mode) on the reinforced concrete columns RCS2G1 (a), RCS2G1W65 (b), RCS2G1W50 (c), RCS2G2W40 (d)

In reinforced concrete columns with S220 steel quality bars (Figure 3.11), the corner confining stress is higher at the strap region for the partial wrapped columns while it is higher at the stirrup region for the full confined columns (as the double confining effect occurs). However a closer investigation of the concrete section slices reveals that the region of the highest confining stress is not at the corner outer node of the section as in plain concrete columns. The analyses suggest that it is located in the region between the longitudinal bar and the outer corner surface. That concrete region of high stress concentration presents values more that twofold than the corresponding in unreinforced columns. Moreover, the highest confining stresses are located at the region below the stirrup and in between the bar and the outer corner surface (figure 3.11). The highest confining stress is observed for 65 mm strap and it is followed by the column with 2 layers of 40 mm strap and by the fully wrapped column. The same is valid for the columns with B500C steel quality bars (Figure 3.12). The highest stress but the analytical stress-strain curve overestimates experimental axial stresses) and it is followed again by the column with 2 layers of 40 mm strap.

Figures 3.13, 3.14, 3.15 show the tensile stress variation on the FRP jackets, plus, the region of highest tensile stresses is arrowed. In plain concrete columns the maximum (failure) tensile stress on the FRP wrapping is located at the middle of the side of the column, at the middle-height of the full wrapped column followed by the FRP tensile stress in 50 mm, 40mm and 65 mm straps. For S1 series columns the utilization of tensile stress of the FRP at failure is higher than in plain columns. The highest tensile failure stress is marked for 65 mm strapped column with S220 bars. The column with 40 mm straps

follows and then the fully wrapped and the 50 mm strapped one. In columns with B500C bars the utilization of stress capacity of the FRP wrap is even higher. The maximum tensile stress on the FRP is developed in the fully wrapped column. Thus in the studied strengthening cases of light FRP partial or full confinement, the presence of the longitudinal bars leads to a higher utilization of the FRP tensile stress capacity. For higher quality bar of the same diameter that utilization is higher.



Figure 3.13. Variation of tensile stresses on FRP jackets of plain columns SG1 (a), SG1W65 (b), SG1W50 (c), SG2W40 (d)



Figure 3.14. Variation of tensile stresses on FRP jackets of reinforced concrete columns RCS1G1 (a), RCS1G1W65 (b), RCS1G1W50 (c), RCS1G2W40 (d)



Figure 3.15. Variation of tensile stresses on FRP jackets of reinforced concrete columns RCS2G1 (a), RCS2G1W65 (b), RCS2G1W50 (c), RCS2G2W40 (d)



Figure 3.16. Variation of maximum tensile stress for different confining schemes in plain and reinforced concrete columns

Figure 3.16 further enlightens the variations in the utilization of the stress capacity of the FRP wraps among different columns. The lower the effectiveness of the confining reinforcement the lower the utilization of the FRP stress capacity. The used longitudinal and transverse reinforcement (despite its sparse detailing) enhances the overall confinement effectiveness on the concrete core and increases the developed FRP stresses. The higher the quality of the bar for the same diameter, the higher the FRP ultimate tensile stresses are.

4. CONCLUSIONS

A 3-dimensional finite element analysis is used to study the mechanical behavior of plain and reinforced concrete columns strengthened by light GFRP full or partial wrapping. A Drucker – Prager type non-associated plasticity is used to simulate the FRP confined concrete triaxial behaviour.

The analytical curves of stress versus axial strain are compared well with the experimental results for all three series of specimens. The fair accuracy of the model enables for the safe investigation of the variations of stresses on concrete and FRP, concerning full and partial confinement.

In plain concrete columns the confining stresses are higher in the region at the corner of the section. Those stresses vary significantly across the height of the member for narrower straps or higher thickness of the straps. The highest corner confining stress is remarked in the FRP strap region of 2 layers of 40 mm strap (almost similar to the one of full confinement).

In reinforced concrete columns with S220 or B500C quality bars, the corner confining stress is higher at the strap region for the partial wrapped columns while it is higher at the stirrup region for the full confined columns (as the double confining effect occurs). Nevertheless, the highest confining stress is located in the region near the longitudinal bar towards the outer corner surface and bellow the steel stirrup (not at the strap level). That concrete region of high stress concentration presents values more that twofold than the corresponding in unreinforced columns. The highest confining stress is observed for 65 mm strap and it is followed by the column with 2 layers of 40 mm strap and by the fully wrapped column.

The maximum (failure) tensile stress on the FRP wrapping is located at the middle of the side of the column, at the middle-height. In plain concrete columns the maximum FRP tensile stress occurs in columns with full wrapping confinement. The presence of the longitudinal bars and stirrups leads to a higher utilization of the FRP tensile stress capacity. The lower the effectiveness of the confining reinforcement, the lower the utilization of the FRP stress capacity. The used longitudinal and transverse reinforcement (despite its sparse detailing) enhances the overall confinement effectiveness on the concrete core and increases the developed FRP stresses. The higher the quality of the bar for the same diameter, the higher the FRP ultimate tensile stresses.

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