Cyclic Response of R/C Jacketed Columns Including Modelling of the Interface Behaviour

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SUMMARY:
The research presented herein aims at the development of an analytical model for predicting the response of members with old-type detailing, strengthened with R/C jacketing under reversed cyclic loading. The analytical model introduces one degree of freedom between the existing member (core of the retrofitted member) and its outer R/C shell, thus allowing slip to take place at the interface between the existing member and the jacket. Shear resistance mechanisms, such as aggregate interlock, friction, and dowel action, are mobilized in response to slip. Constitutive models from the international literature are adopted to describe the mechanisms that resist sliding under cyclic shear reversals. Dual-section analysis is adopted to calculate the shear flow at the interface between the existing member and the jacket. A calculation algorithm is developed to estimate the flexural response under cyclic loading taking into account slip at the interfaces. The validity of the proposed analytical model is assessed through comparison with experiments from the international literature.

Keywords: Reinforced Concrete, Jackets, Retrofitting, Shear transfer, Cyclic loading

1. INTRODUCTION

Earthquakes worldwide have repeatedly demonstrated the vulnerability of the existing building stock, a major source of it being the lack of seismic detailing. This is a rather alarming issue considering the socio-economic impact in case of severely damaged buildings or collapses in future strong ground motions. The viable solution for this category of buildings, which comprises a large part of the existing stock, is retrofitting in order to upgrade their seismic capacity and meet (as far as feasible) the current standards for seismically designed structures. The optimum retrofit strategy may involve a combination of global and/or local intervention measures. If the objective is to provide uniformly distributed lateral load capacity throughout the structure thereby avoiding concentration of seismic damage, then reinforced concrete (R/C) jacketing is arguably the most appropriate intervention method. The addition of the jacket (outer R/C shell) to the existing member increases stiffness and strength, and with proper deign ensures a flexural plastic mechanism, preventing brittle failure modes. Deformation capacity may also be enhanced, depending on the detailing of the existing member.

The response of the composite member is rather complex, thus a pragmatic design approach commonly adopted by codes of practice considers the monolithic approach for the analysis of composite members making use of properly defined 'monolithicity factors' for obtaining the mechanical properties of the strengthened member. The estimation of monolithicity factors is based on empirical or semi-empirical relationships, notwithstanding that design of R/C jacketed members is a complex problem of mechanics that is greatly influenced by the interfacial resistance mechanisms.

An analytical model is developed here for predicting the response of R/C jacketed members taking into account slip at the interface between the existing member and the jacket under cyclic loading conditions. The solution algorithm is based on previous research conducted by Thermou et al. (2007) for monotonic loading. In this paper, it is further modified and extended to account for cyclic shear reversals. The validity of the proposed analytical model was assessed through comparison with experimental moment – curvature histories. An original program was developed based on the proposed solution algorithm, which comprises a useful tool for deriving monolithicity factors.

2. INTERFACE BEHAVIOUR UNDER CYCLIC LOADING CONDITIONS

Shear transfer mechanisms mobilized along interfaces due to slip and their interaction are a rather complex mechanical issue, especially under cyclic loading conditions where degradation should also be accounted for. Mechanisms that resist sliding (slip) are: (i) aggregate interlock between contact surfaces, including any initial adhesion of the jacket concrete on the substrate; (ii) friction owing to clamping action of reinforcement normal to the interface; and (iii) dowel action of any properly anchored reinforcement crossing the sliding plane. The first two mechanisms refer to the contribution of concrete, since they are based on the friction resistance of the interfaces. The relationship that describes the contribution of the individual shear transfer mechanisms is:

$$\begin{aligned} \tau_{tot} &= \tau_{agr} + \tau_f + \tau_D = \tau_{agr} + \mu\sigma_N + \tau_D \Leftrightarrow \\ \tau_{tot} &= \tau_{agr} + \mu(\sigma_c + \rho\sigma_s) + \tau_D = \tau_{agr} + \mu(\nu f_c + \rho\sigma_s) + \tau_D \end{aligned} \tag{2.1}$$

where τ_{agr} represents the shear resistance of the aggregate interlock mechanism, μ is the interface shear friction coefficient, σ_N is the normal clamping stress acting on the interface and τ_D is the shear stress resisted by dowel action in cracked reinforced concrete. The clamping stress represents any normal pressure, p, externally applied on the interface, but also the clamping action of reinforcement crossing the contact plane, σ_s is the axial stress of the bars crossing the interface, ρ is the corresponding reinforcement area ratio, $\nu=N/(A_c f_c)=\sigma_c/f_c$ is the normalized axial load at the interface of area A_c, and f_c is the concrete compressive strength.

2.1. Friction and Dowel Resistance under Cyclic Loading

The only model found in the literature that estimates the combined dowel and shear friction resistances for a given slip value at the interface under cyclic loading is that of Tassios and Vintzileou (1987), Vintzileou and Tassios (1986, 1987). This model was also adopted by the current code for retrofitting in Greece, KANEPE (2012), and is used in this form in the present study, with additional modifications and extensions, as presented in the next section.

The shear stress transferred through friction at the interface is described by the following set of equations (Tassios and Vintzileou 1987):

$$\frac{\tau_f(s)}{\tau_{f,u}} = 1.14\left(\frac{s}{s_u}\right)^{1/3} \qquad \text{for } \frac{s}{s_u} \le 0.5 \tag{2.2a}$$

$$\frac{\tau_f(s)}{\tau_{f,u}} = 0.81 + 0.19\left(\frac{s}{s_u}\right) \qquad \text{for } \frac{s}{s_u} > 0.5 \tag{2.2b}$$

where s_u is the higher value of slip attained (recommended value of 2 mm), whereas the peak value of friction resistance, τ_{fu}, is equal to:

$$\tau_{f,u} = 0.4\left(f_c^2\sigma_N\right)^{1/3} \tag{2.3}$$

which according to KANEPE (2012) depends on the compressive strength of the weakest concrete of the interface, f_c, (typically $f_c=f_{c,old}$). In the experimental study conducted by Júlio et al. (2006), it was found that the increase of the concrete compressive strength of the new layer compared to that of the existing one leads to an enhancement of the compressive strength of the interface, i.e. taking into account the compressive strength of the weakest concrete seems to be conservative. In order to take into account this experimental finding, Eqn. 2.3 was modified here as follows:

$$\tau_{f,u} = 0.4 \cdot \beta\left(f_c^2 \sigma_N\right)^{1/3} = 0.4 \cdot \beta\left(f_c^2 \rho \cdot \sigma_s\right)^{1/3} = 0.4 \cdot \beta \left[f_c^2 \rho \left(\frac{0.3 s^{2/3} E_s f_c}{D_b} \right)^{1/2} \right]^{1/3} \tag{2.4}$$

where parameter β takes into account the increase of the higher value of friction resistance by means of the ratio of the compressive strengths of the new over the old concrete. Hence, β=1.16 if $f_{c,new}/f_{c,old}$=1.0~1.36, β=1.16~1.25 if $f_{c,new}/f_{c,old}$=1.36~2.75 and β=1.25 if $f_{c,new}/f_{c,old} \geq 2.75$. The term σ_N in Eqn. 2.4 has been substituted by the term ($\rho \cdot \sigma_s$) (i.e. dimensionless axial load at the interface was considered, ν=0, see Eqn. 2.1, as in the case of the interface of R/C jacketed members), where σ_s is the steel bar stress at the contact plane which for uniform bond stresses along the embedment length is equal to $(0.3s^{2/3}E_s f_{c,old}/D_b)^{1/2}$. E_s is the elastic modulus of steel and D_b is the dowel diameter (stirrup diameter of the jacket in the case of the proposed model).

According to the degradation rule adopted by KANEPE (2012), the frictional resistance is reduced at each cycle, n, according to:

$$\tau_{f,n} = \tau_{f,1} \cdot \tau_{deg} = \tau_{f,1} \left\{ 1 - \left[0.05(n-1)^{1/2} \left(\frac{f_c}{\sigma_N} \right)^{1/2} \left(\frac{s}{s_u} \right)^{1/3} \right] \right\} \tag{2.5}$$

where $\tau_{f,1}$ is the peak frictional resistance value attained in the first cycle.

In the dowel model proposed by Vintzileou & Tassios (1986, 1987), the bar behaves similarly to a horizontally loaded free-headed pile embedded in cohesive soil, and yielding of the dowel and crushing of concrete occur simultaneously. Dowel force, F_D, is given as a function of slip, s, by:

$$\frac{F_D(s)}{F_{D,u}} = 0.5 \frac{s}{s_{el}} \quad \text{for } s \leq s_{el} = 0.006 D_b \tag{2.6a}$$

$$s = 0.006 D_b + 1.76 s_u \left[\left(\frac{F_D(s)}{F_{D,u}} \right)^4 - 0.5 \left(\frac{F_D(s)}{F_{D,u}} \right)^3 \right] \quad \text{for } \frac{F_D(s)}{F_{D,u}} \geq 0.5 \tag{2.6b}$$

where s_{el} is the elastic slip value, s_u is the ultimate slip value, $F_{D,u}$ is the ultimate dowel force and D_b is the diameter of the dowels (i.e. the legs of the jacket transverse reinforcement). The ultimate dowel strength and associated interface slip are given by:

$$F_{D,u} = 1.3 D_b^2 \left(f_{cd} f_{yd} \right)^{1/2}; \quad s_u = 0.05 D_b \tag{2.7}$$

where f_{yd}(=f_y/1.15) is the design yield strength of steel and f_{cd}(=f_{ck}/1.5) is the design concrete compressive strength. The degradation rule adopted by KANEPE (2012) is:

$$F_{D,n} = F_{D,1} \cdot D_{deg} = F_{D,1} \left[1 - \frac{1}{7} \sqrt{n-1} \right] \tag{2.8}$$

where $F_{D,1}$ is the peak dowel resistance attained in the first cycle and n the number of cycles.

2.2. Modifications to the Interface Constitutive Law

The friction and dowel resistance models presented above were further enhanced to account for the case of non-symmetric reversed cyclic loading (Fig. 2.1) that is typical in seismic situations. Details regarding the new reloading and unloading rules can be found in Papanikolaou et al. (2012).

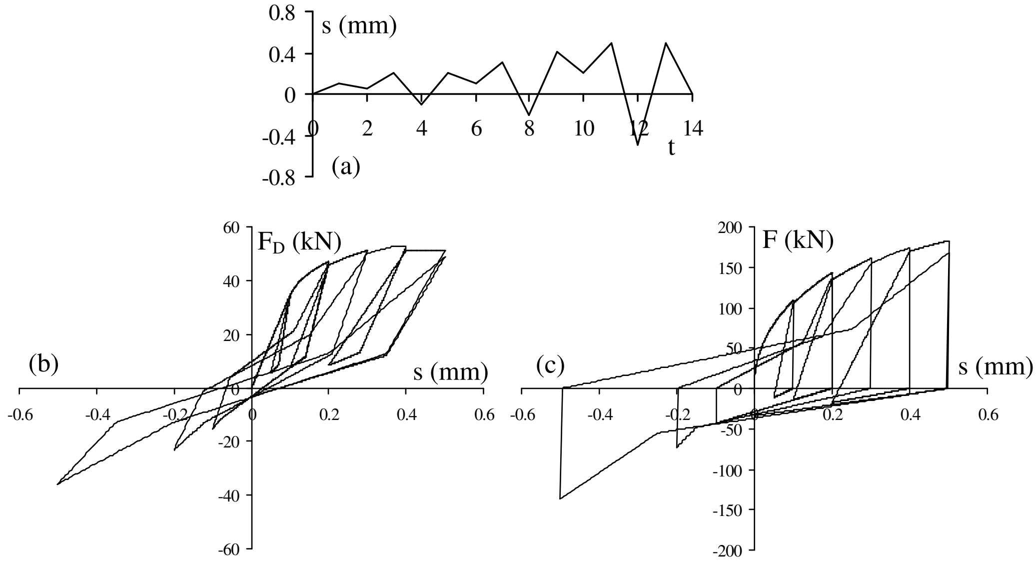

Figure 2.1. (a) Non-symmetric loading history; (b) Dowel resistance hysteretic curve; (c) Total interface resistance hysteretic curve

The degradation rules for friction (Eqn. 2.5) and dowel resistance (Eqn. 2.8) were modified by considering an 'equivalent' number of cycles ($n_{eq}=1/4(\Sigma s/|s_{max}|+1)$) which rather than depending on the number of symmetric cycles (n) as in KANEPE (2012), depends on the cumulative slip (Σs). In addition, a degradation parameter α was introduced allowing the investigation of the sensitivity of the proposed model for R/C jacketed members to different levels of degradation. Eqns. 2.5 and 2.8 where modified as follows:

$$\tau_{deg} = 1 - \alpha \cdot 0.05 \cdot \sqrt{\frac{\sum s/|s_{max}| + 1}{4}} \cdot \left(\frac{f_c}{\sigma_c}\right)^{1/2} \left(\frac{|s_{max}|}{s_u}\right)^{1/3} \tag{2.9}$$

$$D_{deg} = 1 - \alpha \cdot \frac{1}{7} \sqrt{\frac{\sum s/|s_{max}| + 1}{4}} \tag{2.10}$$

where factor α assumes values from "0", which corresponds to null reduction in shear strength, to "1", which corresponds to the degradation described by the rules adopted by KANEPE (2012).

3. DESCRIPTION OF THE PROPOSED ANALYTICAL MODEL

The proposed analytical model for predicting the flexural response of existing R/C members strengthened with concrete jacketing under cyclic loading conditions introduces a degree of freedom allowing the relative slip at the interface between the existing member and the jacket (Thermou et al. 2007). Slip along the member's length is attributed to the difference in normal strains at the contact interfaces (Fig. 3.1(a)). For flexural analysis, the cross-section is divided into three layers which bend with the same curvature, φ (Fig. 3.1(a)). The two external layers represent the contribution of the jacket, whereas the internal one represents both the core (existing cross section) and the web of the jacket shell. Slip at the interface mobilizes the shear transfer mechanisms such as aggregate interlock, friction due to clamping action, and dowel action provided by the stirrup legs of the jacket and by the dowels placed at the interface between the core and the jacket in case that such a connection measure is taken.

According to the analytical model of Thermou et al. (2007) for R/C jacketed members, shear transfer at the interface between the existing member and the jacket takes place between half crack intervals along the length of the jacketed member, as commonly considered in bond analysis. At the initial stages of loading, cracks form only at the external layers (jacket) increasing in number with increasing load, up to crack stabilization (Fig. 3.1(b)). This occurs when the jacket steel stress at the crack, $\sigma_{s,cr}$ exceeds the limit (*fib* 2010):

$$\sigma_{s,cr} > f_{ctm} \frac{1+\eta\rho_{s,eff}}{\rho_{s,eff}} \tag{3.1}$$

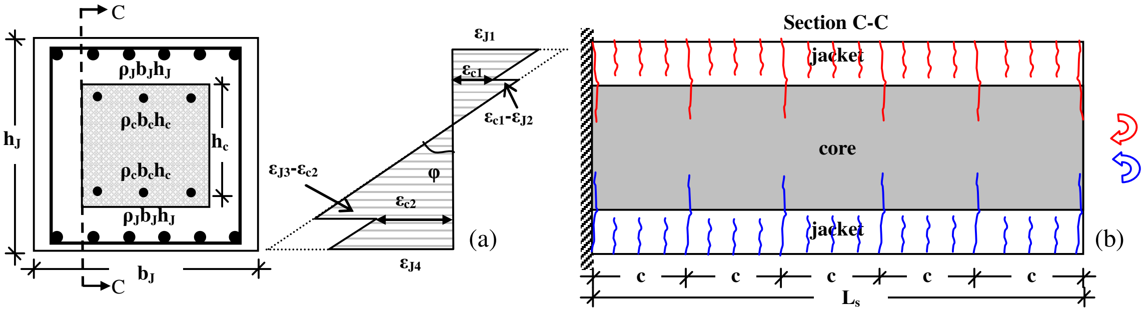

Figure 3.1. (a) Strain profile of the jacketed cross section; (b) Definition of crack spacing, c, at crack stabilization

where f_{ctm} is the tensile strength of concrete, $\eta(=E_s/E_c)$ is the modular ratio and $\rho_{s,eff}$ is the effective reinforcement ratio defined as the total steel area divided by the area of mobilized concrete in tension, usually taken as a circular domain with a radius of $2.5D_b$ around the bar (*fib* 2010). Using the same considerations in the combined section it may be shown that a number of the external cracks penetrate the second layer (core) of the jacketed member (Fig. 3.1(b)). The distance between those cracks, taken as c, is a key element of the proposed methodology (Fig. 3.1(b)).

After crack stabilization and assuming that the neutral axis depth is about constant in adjacent cross sections, from the free body equilibrium in the tension zone of the core of the composite section (Fig. 3.2(a)), the crack spacing is defined as follows (Thermou et al. 2007):

$$c = \frac{0.64 \cdot b_J\, l_c\, f_{ct,c}}{n_c\, D_{b,c}\, f_{b,c} + n_J\, D_{b,J}\, f_{b,J}} \tag{3.2}$$

where b_J is the width of the jacketed cross section, l_c is the height of the tension zone in the core of the composite cross section, $f_{ct,c}$ is the tensile strength of concrete core, n_c, n_J are the number of bars in the tension steel layer of the core and the jacket, respectively, $D_{b,c}$, $D_{b,J}$ are the bar diameter of the core and jacket longitudinal reinforcement, respectively, and $f_{b,c}$, $f_{b,J}$ are the average bond stress of the core and the jacket reinforcement layer, respectively.

Shear stress demand at the interfaces, $\tau_{d,i}$, is determined by examining the cross section along the height and along a member length equal to the distance between successive cracks (Figure 3.2(b)). The layer force resultant ΣF_i (sum of concrete and steel forces at each layer), for the externally applied axial load, N_{ext} (considered to be applied to the jacketed section), is used to calculate the vertical shear stress demand in the member, $\tau_{d,i}$. With the assumption that the shear flow, q, reversal takes place at length equal to c/2 (where c is the crack spacing), the average stress demand $\tau_{d,i}$ is equal to:

$$\tau_{d,i} = \frac{\Sigma F_i}{0.5\, c\, b_J} \tag{3.3}$$

where ΣF_i is the layer force resultant, b_J is the width of the jacketed cross section, and c is the crack spacing length.

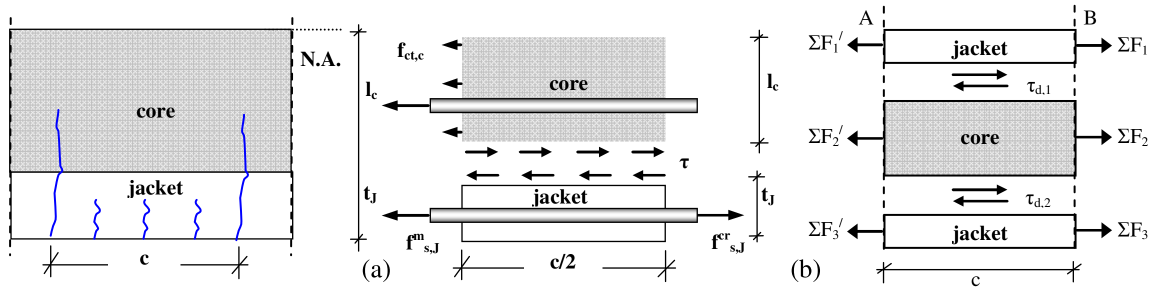

Figure 3.2. (a) Free body equilibrium in the tension zone of the core of the composite section; (b) Section equilibrium between adjacent cracks

4. PRESENTATION OF THE ALGORITHM FOR MOMENT – CURVATURE HISTORIES

The objective of the calculation algorithm at each loading step is twofold; simultaneous establishment of equilibrium between the shear stress capacity and demand at the interfaces for relative slip, s, and force equilibrium at the cross section. An iterative procedure is followed and equilibrium is established until convergence is achieved.

In the first step of the analysis (at very low curvature value) slip is taken equal to zero at both interfaces. In the next steps, as curvature increases, the gradient of the strain profile is modified (allowing continuously increasing difference of strain at the interfaces) in order to establish cross section equilibrium.

For each loading cycle ℓ, in the first step the sectional curvature equal to $\varphi^n(+)$ is set. The unknowns are the normal strain at the top fiber of the jacketed cross section, $\varepsilon_{J1}^{n,m}$, the interface slip at the upper, $s_1^{n,r}$, and bottom interfaces, $s_2^{n,r}$. A value is estimated for the normal strain at the top fiber of the cross section, $\varepsilon_{J1}^{n,m}$ (Fig. 3.1(a)) and the interface slip at the upper and bottom interfaces, $s_1^{n,r}$ and $s_2^{n,r}$, is estimated as the difference of strains at the upper and lower interface, $\Delta\varepsilon_1^{n,r}$ and $\Delta\varepsilon_2^{n,r}$, at length equal to the crack spacing, c, as follows:

$$s_1^{n,r} = \Delta\varepsilon_1^{n,r}\, c = (\varepsilon_{c1}^{n,r} - \varepsilon_{j2}^{n,r})\, c\,,\;\; s_2^{n,r} = \Delta\varepsilon_2^{n,r}\, c = (\varepsilon_{j3}^{n,r} - \varepsilon_{c2}^{n,r})\, c \qquad (4.1)$$

where variables $\varepsilon_{c1}^{n,r}$, $\varepsilon_{j2}^{n,r}$ and $\varepsilon_{j3}^{n,r}$, and $\varepsilon_{c2}^{n,r}$ are normal strains in the section layers above and below the contact surfaces (Fig. 3.1(a)), and c is the average crack spacing (Eqn. 3.2, Fig. 3.2(a)). The next step involves the attainment of equilibrium at the top and bottom interface for slip values, $s_1^{n,r}$ and $s_2^{n,r}$, respectively. Thus, shear strength capacity of the top and bottom interface, , $\tau_1^{n,r}$ and $\tau_2^{n,r}$, estimated according to the constitutive laws that describe the behaviour of the interface under cyclic loading (Eqns. 2.1-2.9) and shear strength demand at the upper and bottom interface, $\tau_{d,1}^{n,r}$ and $\tau_{d,2}^{n,r}$, estimated according to Eqn. 3.3 should be equal. If this holds, then equilibrium is established and the next step follows, otherwise slip values at the top and bottom interface need be revised till convergence. The values to be set for the top and bottom slip are $s_1^{n,r+1}=s_1^{n,r}+ds_1$, $s_2^{n,r+1}=s_2^{n,r}+ds_2$, where ds_i is the selected increment in the slip value. After the necessary iterations till equilibrium at the interfaces is established, the cross section should also be in equilibrium. The force resultant, ΣF_i (Fig. 3.2(b)) is calculated at each layer. In case that equilibrium is not established, $(\Sigma\Sigma F_i\text{-}N_{ext})\geq$tolerance, then the normal strain profile is revised by setting $\varepsilon_{J1}^{n,m+1}=\varepsilon_{J1}^{n,m}+d\varepsilon_J$, where $d\varepsilon_J$ is the step increment in the top strain of the jacketed cross section. The convergent values for which equilibrium at both interface and cross section level was established ($\varepsilon_{J1}^{n}=\varepsilon_{J1}^{n,m}$, $s_1^{n}=s_1^{n,r}$, $s_2^{n}=s_2^{n,r}$) are stored and the moment resultant, M^n is estimated. The algorithm enters the unloading phase and the whole procedure described above is repeated for $\varphi^n(-)$. The algorithm enters in the unloading phase and all steps are repeated for $\ell=\ell+1$. Calculations terminate when the shear capacity of the interface is exhausted.

Due to the complexity of the proposed solution algorithm, a program was necessary to be developed where fiber analysis was considered. The stress – strain envelope and the hysteretic rules describing the behaviour of concrete followed the model of Mander et al. (1988) and Martinez-Rueda and Elnashai (1997), respectively. The constitutive law for steel was based on the stress-strain relationship developed by Menegotto-Pinto (1973) in combination with the rules of isotropic hardening proposed by Fillippou et al. (1983). The constitutive laws for the description of the shear transfer mechanisms mobilized due to slip at the interface are based on the models of Tassios and Vintzileou (1987), Vintzileou and Tassios (1986, 1987), KANEPE (2012) modified as described in Section 2.2. More details regarding the computational procedure can be found in Papanikolaou et al. (2012).

5. VALIDATION OF THE PROPOSED ANALYTICAL MODEL

The validity of the proposed analytical model for predicting the flexural response of R/C jacketed members under cyclic loading conditions was examined by comparing the moment – curvature histories derived by the analytical model with the corresponding experimental ones. The sensitivity of the proposed model to the degradation rules adopted after the Greek code for interventions KANEPE (2012) (Eqns. 2.5, 2.8) as modified for the needs of current study (Eqns. 2.9, 2.10) was examined. The results of the sensitivity analysis were implemented in the analytical model in order to derive the moment – curvature hysteretic curves of a group of test specimens. It was shown that the degradation rules adopted by the code (KANEPE, 2012) are rather conservative and that by employing lower values for degradation factor α (Eqns 2.9, 2.10) the derived hysteretic curves match the experimental ones.

5.1. Sensitivity of the Analytical Model to Degradation Rules

Specimen QRC was selected from the experimental work of Bousias et al. (2007) for a parametric study addressing only the factor "α" which determines the reduction of shear stress (Eqns. 2.9, 2.10). Specimen QRC has a 250 mm square cross section where a 75 mm thickness jacket is added with shear span L_s=1.6 m. No special measures were taken to connect the jacket to the existing cross section (core). Details regarding reinforcement detailing, material properties and axial load appear in the experimental database created for R/C jacketed members by Thermou et al. (2011).

The curvature loading history of Fig. 5.1 was applied to the analytical model of QRC for various values of degradation factor α (α=0.2, 0.4, 0.6, 0.8, 1.0). In Fig. 5.2, experimental data is compared with the moment curvature histories for the selected cases of α=1.0 and α=0.4. The first case represents the response according to the degradation rules adopted by KANEPE (2012) as modified in Eqns. 2.9, 2.10. The case where α=0.4 leads to hysteretic response curves that are closer to the experimental results.

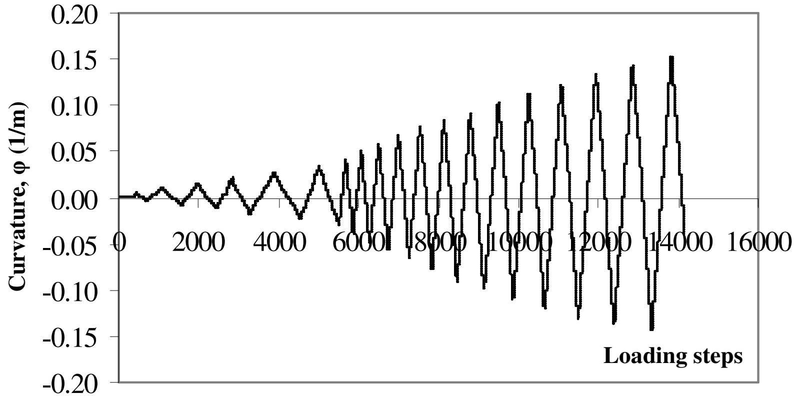

Figure 5.1. Curvature loading history

From the above, it seems that the degradation rules proposed by KANEPE (2012) are conservative, whereas adopting a value of α=0.4 in the analytical model the derived moment - curvature histories are close to the experimental ones.

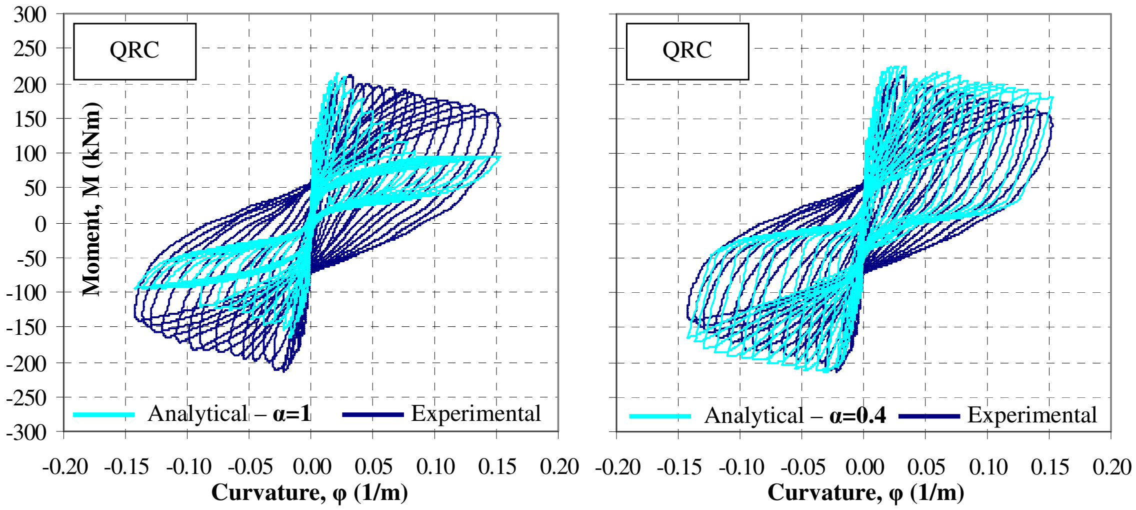

Figure 5.2. Comparison of moment – curvature histories between experimental data and analytical results based on the proposed analytical model for R/C jacketed members for specimen QRC (Bousias et al. 2007)

5.2. Experimental Validation

The experimental moment – curvature histories for the group of specimens studied by Bousias et al. (2007) were compared with the moment – curvature histories derived according to the analytical model. All specimens had the same geometry and the same jacket thickness as specimen QRC described above (more details can be found in the database compiled by Thermou et al. 2011). Specimen QRCM was a control specimen built monolithically having the same geometry with the jacketed members, its only longitudinal reinforcement being that of the jacket. Apart from specimen QRC for which no special measures were taken to connect the existing member to the jacket, for all the other specimens various connection measures were examined. In specimen QRCR, the full lateral surface of the existing column was roughened using a pneumatic chipping device, and in QRCD three 16 mm dowels were driven into each side of the existing column, at distances of 200, 650, and 1100 mm from the top of the footing. In QRCRD the measures taken in QRCR and QRCD were combined. In Q-RCW the corner bars of the jacket were connected to the corresponding existing ones by welding both of them to 16 mm diameter, 400 mm long deformed reinforcing bars bent into a U-shape.

The comparison between the experimental and the analytical moment – curvature histories for the same loading history (Fig. 5.1) are presented in Fig. 5.3. In the case of the monolithic specimen, QRCM, the response of the analytical model (it is noted that no slip takes place at the interface since a monolithic cross section is examined) matches the secant stiffness at yield, maximum strength and pinching of the hysteretic curves, but fails to follow the strength degradation for curvature values higher than 0.05 m^{-1}. The proposed analytical model cannot take into account various alternative connection measures, as for example the ones applied in the test specimens by Bousias et al. (2007), apart from the case of dowels connecting the existing member and its outer shell (the case of specimens QRCD and QRCRD). For this reason modelling of all test specimens examined herein was made neglecting any connection measures. This decision was supported by the outcome of the experimental work of Bousias et al. (2007), which has shown that key properties such as yield moment, yield drift, secant stiffness at incipient yielding, and flexure-controlled ultimate drift, do not systematically depend on the type of the connection measures taken. An exception to this observation refers to the ultimate chord rotation which increased by approximately 16% in case that the connection measures are dowels or U-bars welded between the new and the existing longitudinal bars. In the light of the above, it seems that the different measures of connection between the existing member and the jacket have minor impact on the response of the jacketed member. This finding can justify the selection of a unique value for degradation factor α for the various connection measures taken. The moment – curvature hysteretic curves for specimens QRC, QRCR, QRCD, QRCRD and QRW were estimated utilising the analytical model, adopting α=0.4 (Eqns. 2.9, 2.10) based on the preceding sensitivity study. The comparative results of Fig. 5.3 indicate that independently of the connection measures taken, a common value for degradation factor α may apply. It is observed that the analytical curves reasonably match the strength and stiffness level of the experimental curves at each loading step. Pinching is more intense in the case of the analytical model (red-coloured hysteretic curves compared to the blue-coloured ones) indicating less energy dissipation at each loading cycle, which is deemed conservative.

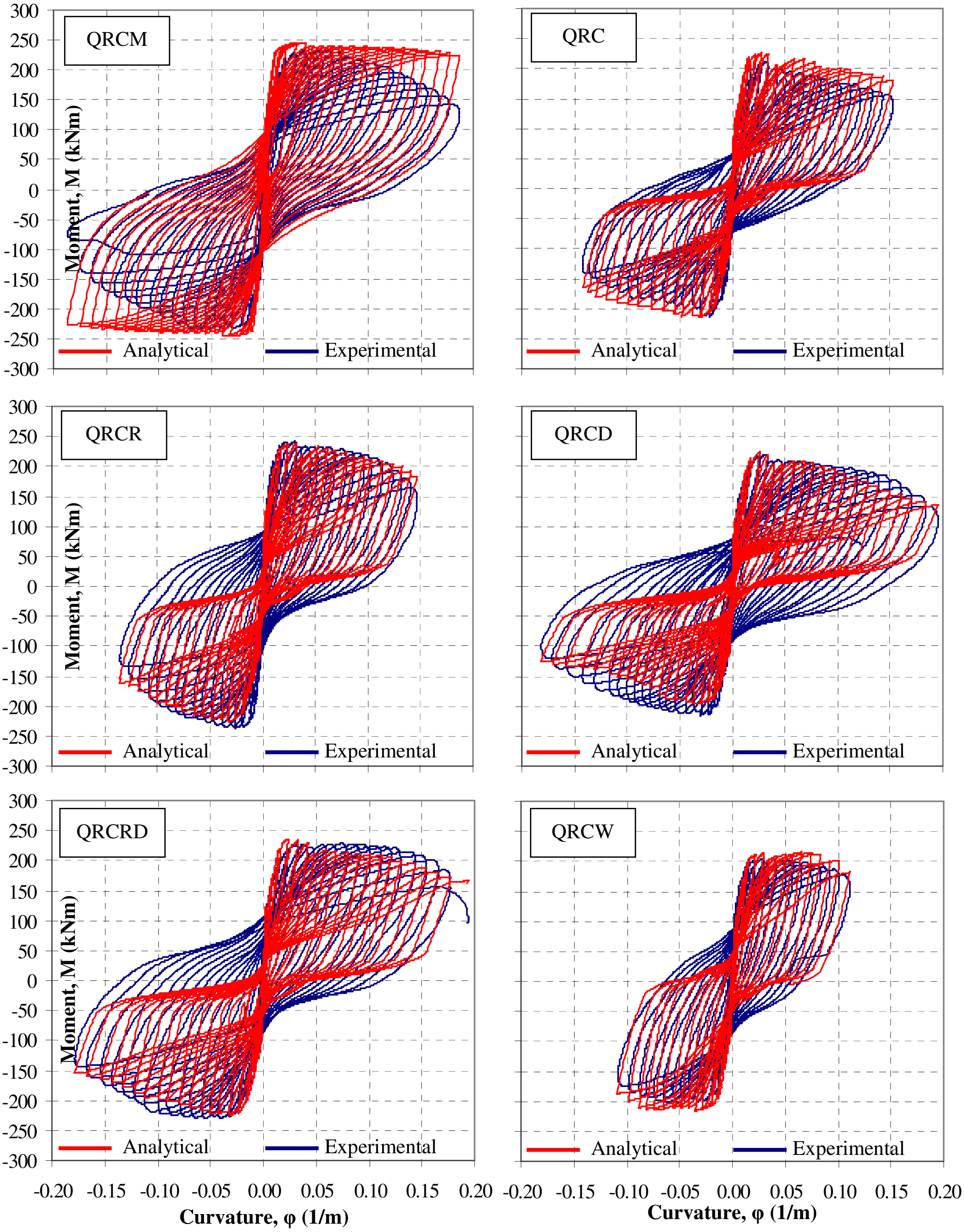

Figure 5.3. Comparison of moment – curvature histories between experimental data and analytical results based on the proposed analytical model for R/C jacketed members

6. SUMMARY AND CONCLUSIONS

The response of old-type R/C columns strengthened with concrete jacketing under reversed cyclic loading was studied in this paper. In the developed analytical model, it is considered that there is partial connection between the core and the jacket and that slip can take place at the interfaces. The composite cross section is divided into three layers which develop the same curvature. The interface characteristics play a crucial role in the response of the composite member. The proposed algorithm aims at establishing equilibrium at the interfaces, which is achieved when shear capacity equals shear demand. Shear capacity at each loading step is described by constitutive models for cyclic loading conditions from the literature, which are further modified and enhanced for the needs of the present research. Shear demand at the interfaces is estimated by considering the flexural stresses on each layer of the cross section and is controlled by the distance between successive cracks. A parametric study was conducted to investigate the sensitivity of the degradation rules adopted. It was shown that a reduced degradation factor leads to analytical moment – curvature histories that correlate well with experimental data. The value of the reduced degradation factor was utilized in deriving the moment – curvature histories of a group of test specimens where different connection measures were taken. The derived analytical curves reasonably matched the experimental ones, thus manifesting the validity of the proposed analytical model for R/C jacketed members under cyclic loading.

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REFERENCES

Bousias, S., Biskinis, D., Fardis, M. and Spathis, A. (2007). Strength, stiffness, and cyclic deformation capacity of the concrete jacketed members. *ACI Structural Journal* **104:5**, 521-531.

fib (2010). Model Code 2010 - First complete draft - Vol. 2, *fib Bull. 56*, Lausanne.

Filippou, F.C., Popov, E.P. and Bertero, V.V. (1983). Effects of bond deterioration on hysteretic behaviour of reinforced concrete joints. *Report No. UCB/EERC-83/19*, University of California, Berkeley.

Júlio, E.N.B.S., Branco, F.A. B, Silva, V.D and Lourenço, J.F. (2006). Influence of added concrete compressive strength on adhesion to an existing concrete substrate. *Building and Environment* **41**, 1934-1939.

KANEPE (2012). Interventions Code (of Greece). *Earthquake Planning and Protection Organization (E.P.P.O.)*, Final draft, Sep. 2012 (in Greek).

Mander J.B., Priestley M.J.N. and Park R. (1988). Theoretical stress-strain model for confined concrete. *Journal of Structural Engineering* **114:8**, 1804-1826.

Martinez-Rueda J.E. and Elnashai A.S. (1997). Confined concrete model under cyclic load. *Materials and Structures* **30:3**, 139-147.

Menegotto, M. and Pinto, P.E (1973). Method of analysis for cyclically loaded RC plane frames including changes in geometry and nonelastic behaviour of elements under combined normal force and bending. *Proceedings IABSE Symposium*, Lisbon, Portugal.

Papanikolaou, V.K., Thermou, G.E. and Kappos, A.J. (2012). Moment-curvature analysis of R/C jacketed rectangular sections including interface slip under cyclic loading. *15th World Conference on Earthquake Engineering*, Lisbon, Portugal.

Tassios, T. and Vintzileou, V.E. (1987). Concrete-to-concrete friction. *Journal of Structural Engineering* **113:4**, 832-849.

Thermou, G.E., Pantazopoulou, S.J. and Elnashai, A.S. (2007). Flexural behavior of brittle RC members rehabilitated with concrete jacketing. *Journal of Structural Engineering* **133:10**, 1373-1384.

Thermou, G.E., Papanikolaou, V.S, and Kappos A.J. (2011). Analytical model for predicting the response of old-type columns rehabilitated with concrete jacketing under reversed cyclic loading. *COMPDYN 2011*, Corfu, Greece. Paper No CD 137.

Vintzileou, E., and Tassios, T. (1986). Mathematical models for dowel action under monotonic and cyclic conditions. *Magazine of Concrete Research* **38:134**, 13-22.

Vintzileou, E., and Tassios, T. (1987). Behavior of dowels under cyclic deformations. *ACI Structural Journal* **84:1**, 18-30.