

# NONLINEAR SEISMIC BEHAVIOR OF AN RC FRAME REPAIRED BY FRP LAMINATES

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### SUMMARY

Strengthening reinforced concrete (RC) frames and unreinforced masonry (URM) infill walls with fiber reinforced polymer (FRP) has been proved a very successful procedure. Although there are numerous researches on the behavior of FRP-strengthened RC columns and beams as well as URM infills, few investigations have been made on the overall effect of FRP strengthening on RC frames with URM infill walls. This paper discusses the effects of using different amounts of FRP on RC frames with various configurations including the number of stories, spans, and locations of infills and so on. The infill walls were modeled according to the equivalent strut model and the effect of strengthening was considered as a tensional strut crossing the compressive strut representing the URM infill wall. The behavior of the strengthened RC columns and beams were modeled according to [Lam and Teng, 2003]. The models were then subjected to push-over loading and the improvement in their behavior is investigated.

## **KEYWORDS**

RC frames, URM infill walls, FRP strengthening, push-over analysis, story drift

## **INTRODUCTION**

In recent years, and from experiences of poor behavior of buildings designed according to codes' regulations happened during past earthquakes, building standard codes have increased their design requirements. These new regulations are mainly aimed to reduce damage of newer buildings to acceptable levels in the event of a moderate to strong earthquakes. However, older buildings, which were designed by codes that are now known to provide inadequate safety, are likely to be vulnerable to severe damage or collapse under strong seismic excitation. Past earthquakes have demonstrated that these older buildings would have survived, in most cases, with a reasonable upgrading. The main drawbacks of the older RC buildings are insufficient lateral stiffness, low ductility and inadequate load-bearing capacity [Ghobarah et al., 2000]. These results in brittle shear failures especially at the joints, soft story and column sideways collapse [Bracci et al., 1995]. On the other hand, according to extensive researches and from past experiences in different earthquakes, masonry infills play a significant role in load-bearing action during earthquake. They influence the overall behavior of buildings in terms of considerably increasing the strength and stiffness and energy dissipation capacity [Moghaddam et al., 1987, Mander et al., 1993, FEMA 306, Decanini et al., 2002]. But in many standard codes, including Iranian code of practice for seismic resistant design of buildings, their effect is underestimated if not ignored. Since infills tend to show a brittle behavior at the final loading stages, a retrofit procedure capable of enhancing ductility of infills is more preferable. Among different strengthening/retrofitting techniques URM infills, using FRP jacketing for RC columns and beams and FRP strips or laminates for URM infills seems one of the best techniques. This is mainly because using FRP is quicker to implement, adds no weight to the existing structures, has very little aesthetic impact and is corrosion resistant [Teng et al., 2002, 2003, Xiao, 2004]. The lateral drift performance of a multi-story building is an important indicator that measures the level of damage to the structural and non-structural components of buildings [Ghobarah et al., 2000, SEAOC, 1995, ATC-40, 1996, FEMA 274, 1997, Moehle, 1991]. Lateral drift design is particularly challenging as it requires the consideration of an appropriate stiffness distribution of all structural elements and, in a severe seismic event, also the occurrence and redistribution of plasticity in the elements [Zou et al., 2006].

### DESCRIPTION OF THE MODEL AND ASSUMPTIONS

In order to study the effects of strengthening infills and frames with FRP, models which had been investigated experimentally were selected which is a 3-bay, 4-story building [Nakano et al., 2004]. The specifications of beams are as follows:  $80 \times 60 cm$ , 10-D29mm, D13mm@100mm. the columns are  $40 \times 45 cm$ , 12-D22mm, D10mm@30mm in the first and the second stories and  $40 \times 45 cm$ , 8-D19mm, D10mm@30mm in the third and the forth stories. The modulus of elasticity and the compressive strength of concrete are 21 GPa and 27.3 MPa, respectively. The steel used as reinforcement has modulus of elasticity equal to 180 GPa and its yield strength is 400 MPa.

According to FEMA 356-6.5.2.2.2 for modeling beams and columns in non-linear static analysis, plastic hinges should be taken into consideration. In order to capture the exact nonlinear behavior of the model by using simple modeling, the commercial software SAP2000 was used and local P-M hinges were assigned in the first and the last 10% of the columns' length. Besides that, in the infills and FRP strips, two axial-load hinges were assigned at aforementioned locations. The yield force for axial-load hinges should be calculated as follows:

- 1) For corner crushing in the infill:  $F_{ult-c} = A_{strut}\sigma_{c-c}$  (1)
- 2) For bed joint sliding in the infill:  $F_{ult-t} = Sec\theta \times \tau_b \times l_m \times t_m$  (2)
- 3) For stepped diagonal cracking in the infill:  $F_{ult-t} = N_h \times \sigma_{t-h} \times h_b + N_b \times \tau_b \times l_b$  (3)

In which  $F_{ult-c}$  and  $F_{ult-t}$  are the yield force of infill axial-load hinges in compression and tension, respectively,  $A_{strut}$  is the cross sectional area of the infill which is modeled as a diagonal strut,  $\sigma_{c-c}$  is the two-axial compressive strength of the masonry infill,  $\theta$  is an angle that satisfies  $\theta = \tan^{-1}(\frac{h_m}{l_m})$  in which  $h_m$  and  $l_m$  are

the height and the length of the infill wall,  $\tau_b$  is the shear strength of the bed joints,  $t_m$  is the thickness of the infill,  $N_h$  and  $N_b$  are the number of head and bed joints adjacent to stepped-diagonal cracks,  $\sigma_{t-h}$  is the tensional strength of the head joints, and finally  $h_b$  and  $l_b$  are the height and the length of the masonry blocks, respectively.

It is noteworthy that in many regions in Iran, masons put no mortar in the head joints and call this method as "Khoshkeh Chini", hence  $\sigma_{t-h} = 0$ . Besides that, since the head joints of bricks are smoother than the bed joints; their frictional coefficient can be ignored. Furthermore, in practical and design aspects, infill struts are modeled as compression-only elements; so, engineers assume that only corner-crushing mode of failure is the matter of importance. This assumption seems logical when reviewing academic researches [Ghazimahelleh, 2007, Akbari, 2006, and many others]

For modeling strengthening of columns with FRP laminates, the only parameter which can be taken into account in simplified macro modeling is improving the stress-strain curve of RC used in the frames. In doing so, the model proposed by [Lam and Teng, 2003] was selected. Not only the model is very simple, but also it can capture the behavior of RC columns without FRP. The outline of Lam and Teng's model is depicted on Figure 1.



Figure 1 Lam and Teng model for concrete strengthened with FRP

The formulation of Lam and Teng model is as follows:

$$\sigma_c = E_c \varepsilon_c - \frac{(E_c - E_2)^2}{4f_c} \varepsilon_c^2 \qquad \text{for } 0 \le \varepsilon_c \le \varepsilon_t$$
(4)

$$\sigma_{c} = f_{co}' + E_{2}\varepsilon_{c} \qquad \qquad \text{for } \varepsilon_{t} \le \varepsilon_{c} \le \varepsilon_{cu} \tag{5}$$

Where

$$E_2 = \frac{f'_{cc} - f'_{co}}{\varepsilon_{cu}}$$
 and  $\varepsilon_t = \frac{2f'_{co}}{E_c - E_2}$  (6),(7)

In these equations,  $\sigma_c$  and  $\varepsilon_c$  are the axial compressive stress and strain of confined concrete,  $E_c$  and  $f'_{co}$  are the initial modulus of elasticity and the compressive strength of unconfined concrete, respectively;  $E_2$  is the slope of the straight line that intercepts the stress axis at  $f_o = f'_{co}$ ,  $\varepsilon_t$  is the axial strain of concrete at which the parabolic first portion meets the linear second portion with a smooth transition. Also,  $f'_{cc}$  and  $\varepsilon_{cu}$  are the compressive strength and ultimate compressive strain of FRP-confined concrete and can be obtained from:

$$\frac{f_{cc}}{f_{co}} = 1 + 3.3k_{s1}\frac{f_1}{f_{co}} \quad \text{and} \quad \frac{\varepsilon_{cc}}{\varepsilon_{cu}} = 1.75 + 12k_{s2}(\frac{f_1}{f_{co}})(\frac{\varepsilon_{h,rup}}{\varepsilon_{co}})^{0.45}$$
(8),(9)

In which  $\varepsilon_{cu}$  is the axial strain at peak stress of unconfined concrete, taken as 0.002 [Lam and Teng, 2003].  $\varepsilon_{h,rup}$  is the FRP hoop rupture strain;  $f_1$  is the equivalent confining pressure,  $k_{s1}$  and  $k_{s2}$  are the shape factors for strength and ultimate strain, respectively. These parameters can be derived according to the following equations:

$$f_1 = \frac{2E_{frp}t\varepsilon_{h,rup}}{\sqrt{(B^2 + D^2)}}, \quad k_{s1} = (\frac{B}{D})^2 \frac{A_e}{A_c} \quad \text{and} \quad k_{s1} = (\frac{D}{B})^{0.5} \frac{A_e}{A_c}$$
(10),(11),(12)

Where  $E_{frp}$  and t are the elastic modulus and the thickness of FRP, respectively. B and D are the width and the height of the section, respectively.  $A_e$  is the area of the effectively-confined concrete and  $A_c$  is the total area of concrete enclosed by the FRP jacket and can be calculated as:

$$\frac{A_e}{A_c} = \frac{1 - \frac{B}{D} (D - 2R_c)^2 + \frac{D}{B} (B - 2R_c)^2}{3A_g} - \rho_{sc}}{1 - \rho_{sc}}$$
(13)

where  $A_g$  is the gross area of the section;  $\rho_{sc}$  is the cross-sectional area ratio of longitudinal steel; and  $R_c$  is the radius of the rounded corners which is recommended to delay the tensile rupturing of FRP and improving the confinement. It should be noted that Lam and Teng's model is valid if only the following eaquation is satisfied:

$$\frac{k_{s1}f_1}{f_{co}} \ge 0.07 \tag{14}$$

The ultimate moment and curvature of FRP-confined rectangular cross-sections can be calculated as:

$$M_{u} = \alpha f_{co}^{'} BX(\frac{D}{2} - \gamma X) + \sum_{k=1}^{N_{s}} f_{sk} A_{sk}(\frac{D}{2} - d_{sk}) \quad \text{and} \quad \phi_{u} = \frac{\varepsilon_{u}}{X}$$
(15),(16)

In which  $f_{sk}$  and  $A_{sk}$  are the stress (positive if compressive) and the cross-sectional area of the  $k^{th}$  layer of steel reinforcement, respectively, and Ns is the number of layers of steel reinforcement. The steel of the  $k^{th}$  layer has a strain  $\varepsilon_{sk}$  and a distance  $d_{sk}$  of its centroid from the extreme compression fiber. X is the neutral axes depth which can be calculated based on the equilibrium of forces and a MATLAB program was written for it by the authors:

$$P = \alpha f_{co}^{'} BX + \sum_{k=1}^{N_s} f_{sk} d_{sk}$$
(17)

 $\alpha$  and  $\gamma$  are the parameters which can be derived as:

$$\alpha = 1 + \frac{E_2 \varepsilon_{cu}}{2f_{co}} - \frac{2f_{co}}{3(E_c - E_2)\varepsilon_{cu}} \text{ and}$$

$$\gamma = 1 - \frac{-2f_{co}^{'3} + 3f_{co}^{'}\varepsilon_{cu}^2(E_c - E_2)^2 + 2E_2\varepsilon_{cu}^3(E_c - E_2)^2}{-4f_{co}^{'2}\varepsilon_{cu}(E_c - E_2) + 3E_2\varepsilon_{cu}^3(E_c - E_2)^2 + 6f_{co}^{'}\varepsilon_{cu}^2(E_c - E_2)^2}$$
(18),(19)

Both infills and FRP strips were modeled as a diagonal strut and with pinned ends. The FRP strips are 200 mm in width and 2.5 mm in thickness and since two layers were use at both sides of the infill wall, the thickness was doubled in the analysis. The modulus of elasticity and the compressive strength of masonry are 12.2 GPa and 17.5 MPa, respectively. For the FRP strips, the Young modulus is 230 GPa and the modulus of rupture is 3430 MPa.

The distributed dead load equal to 88.88 KN/m was exerted on the beams. The push-over analysis was conducted on the model in transverse direction. The control node for displacement is the middle axis of the  $4^{th}$  story. The loading mechanism is according to the  $1^{st}$  mode pattern in which the loading was distributed in triangular shape, with the load equal to the weight of the story in the  $4^{th}$  story.

#### ANALYSES RESULTS

In order to calibrate the simple model and use input data which can lead to correct results, 4 specimens representing one-bay, one-story RC frames with and without URM infill walls located in the 1<sup>st</sup> and the 4<sup>th</sup> stories, and a frame located at the1st story with half infill wall which had been experimentally investigated by [Nakano et al., 2004] are modeled with SAP2000 and Ansys. The details and assumptions of the Ansys model is presented in the parallel paper by authors [Dastan, Yekrang nia and Vafaei, 2008] The calibration process mainly focused on the hinges' behavior. Figure 2 (a) and (b) shows the outline of the experimental and numerical frame specimens.



Figure 2 frame specifications (a) experimental (b) numerical

In columns and struts of the 3<sup>rd</sup> and the 4<sup>th</sup> stories, the hinges were assumed to be deformation-controlled according to FEMA 356, 2000, which is depicted in Figure 3.



Figure 3 hinge behavior in columns and 3<sup>rd</sup> and 4<sup>th</sup> story struts

In deformation-controlled hinges, the vertical axis is the ratio of moment generated at hinge location to the yield moment which can be calculated by Equation (21) mentioned in FEMA 356, 2000, Formula 5-3. Accordingly, the horizontal axis is the ratio of hinge rotation to the yield rotation which can be derived by Equation (20) mentioned in FEMA 356, 2000, Equation 5-2.

$$\theta_{y} = \frac{ZF_{ye}l_{c}}{6EI_{c}} \left(1 - \frac{P}{P_{ye}}\right) \tag{20}$$

$$Q_{CE} = M_{CE} = 1.18ZF_{ye}(1 - \frac{P}{P_{ye}}) \le ZF_{ye}$$
(21)

In which E is modulus of elasticity,  $F_{ye}$  is expected yield strength of the material, I is moment of inertia,  $l_c$  is column length,  $M_{CE}$  is expected flexural strength, P is axial force in the member at the target displacement for nonlinear static analyses,  $P_{ye}$  is expected axial yield force of the member =  $A_g F_{ye}$ ,  $Q_{CE}$  is generalized component expected strength, and Z is plastic section modulus.

There are also recommended amounts for modeling parameters and numerical acceptance criteria for nonlinear procedures for RC and RC Infilled Frames in Tables 6-8 and 6-16 of FEMA 356, 2000, respectively. Furthermore, there is also suggested behavior for nonlinear static procedure of simplified force-deflection relations for masonry infill panels in Table 7-9 of FEMA 356, 2000. Since in this article, the selected models had been experimentally analyzed, there is no need to use the recommended amounts for defining hinges and these parameters were calibrated until finally acceptable results were obtained.

In order to model the URM infill wall, this element can be modeled as a diagonal strut with the material properties the same as masonry and the thickness equal to the thickness of the real URM infill wall. There are some researches on determination of the effective width of masonry strut [El-Dakhakhni et al., 2001, Drysdale et al., 1994, Mainstone and Weeks, 1970, Stafford Smith and Carter, 1969, Stafford Smith, 1967, 1966, 1962, Holmes, 1961]. Since they are simplified models which consider only one mode of failure of the system and do not take into account other possible failure modes, they lead to very different results and in some cases; they can underestimate or overestimate the actual capacity of the infilled frame up to 190% [Parsa, 2006]. In this article, Z is the effective width of the strut which was proposed by [Stafford-Smith and Carter, 1969]. According to FEMA 306, 1998 modifications which were approved in FEMA 356, 2000:

$$Z = 0.175(\lambda h)^{-0.4} d_m, \lambda = \left(\frac{E_m tsin(2\theta)}{4E_c I_g h_m}\right)^{\frac{1}{4}}$$
(22),(23)

H is the column height between centerlines of beams,  $h_m$  is the height of infill panel,  $E_c$  is the expected modulus of elasticity of frame material,  $E_m$  is the expected modulus of elasticity of infill material,  $I_g$  is the moment of inertial of column,  $D_m$  is the diagonal length of infill panel, T is the thickness of infill panel and equivalent strut, and  $\theta$  is the angle whose tangent is the infill height-to-length aspect ratio is as:

$$\theta = \tan^{-1}(\frac{h_m}{l_m})$$

where,  $l_m$  is the length of infill panel

For the one-bay, one-story infilled frames, the effective strut width is 0.42m. In order to check the accuracy of the proposed formula, the aforesaid infilled frames were simulated in Ansys and according to a proper spectra of the 1<sup>st</sup> principal stress contour, the effective strut width was calculated. Figure 4 (a) and (b) shows the distribution of the 1<sup>st</sup> principal stresses at 1cm top displacement.



Figure 4 1<sup>st</sup> principal stresses at 1cm top displacement (a) 1<sup>st</sup> story (b) 4<sup>th</sup> story

Figure 5 (a) and (b) shows the numerical results for the  $1^{st}$  and the  $4^{th}$  stories, respectively. The behavior of infilled frames of the  $1^{st}$  and the  $4^{th}$  stories is shown in Figure 6 (a) and (b), respectively. Figure 7 represents the behavior of the half infilled frame at  $1^{st}$  story. And finally, Figure 8 shows the results of the building.







Figure 6 base shear vs. top displacement for infilled frames in (a) 1<sup>st</sup> story (b) 4<sup>th</sup> story



Figure 7 base shear vs. top displacement strengthened infills with and without frame strengthening (a) 1<sup>st</sup> story (b) 4<sup>th</sup> story



Figure 8 base shear vs. top displacement of the building

As can be seen from the above figures, the simplified macro model with SAP2000 can lead to acceptable results with some calibrations in hinge properties. In these figure, curves labeled FEMA356 are the ones in which the hinge properties have been left the values FEMA356 recommended. It is obvious from the figures that adding FRP to the infill and the framecan greatly enhance the ductility as well as the strength. It is noteworthy that if the infill walls have been strengthened with FRP, strengthening of the frames has negligible influence on the overall behavior of the system. Also adding FRP to the infill has greater effect on strength improvement of the infilled frames in 4<sup>th</sup> story than 1<sup>st</sup> one.

### CONCLUSIONS

In this article, the effect of considering URM infill walls on the overall behavior of RC frames, as well as strengthening both structural elements with FRP was investigated. Great improvements in lateral-load capacity were observed by considering URM infills as a structural element. It was also concluded that using FRP on URM infills as well as RC columns can enhance the ductility of the system which is considered the main drawback of infilled frames. Good agreements between experimental and numerical analyses were obtained.

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