ANALYTICAL COMPARISON OF THE ORDINARY AND HIGH STRENGTH STEEL REINFORCED CONCRETE FRAMES AGAINST EARTHQUAKES

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ABSTRACT: Failure mechanisms in ordinary RC frames can be prevented by adopting strong column weak beam philosophy. Limited flexural strength and lateral deformation capacity of the RC columns often results in the formation of failure mechanisms. Large inelasticity at the column base can be prevented by providing high strength reinforcements in columns. Two bays three story frames reinforced with high strength and ordinary reinforcements in columns are modeled on MSC.MARC finite element code. For section behavior fiber model THUFIBER is used. Bond failures between steel and concrete and slip at the column base are ignored. Modeled frames are also tested. Simulated and the test results are compared. Test results on two bays three story frames with high strength reinforcement in columns and ordinary reinforcements in beams reveals more stable response at large lateral displacements. With the simple replacement of ordinary steel in the column of RC frames, passive frame mechanism can be achieved. Difference between analytical and test results can be realized by recognizing the basic assumptions behind the formulation of fiber model. Nevertheless, the observed and modeled behavior has been found in good agreement.

Keywords: Earthquake; Failure mechanisms; Fiber model; Frames; High strength reinforcement; Response.

INTRODUCTION

Failure of modern structures under most recent earthquakes such as Kobe, Northridge and Kashmir has exposed weakness in the current design techniques and philosophies. In the performance based design, structures are designed for the desired performance target under the probable dynamic event. To regain the equilibrium position and to minimize the residual drift is the desirable response after strong ground motions. However, due to limited flexural strength of ordinary steel reinforced columns innovative materials and techniques have been studied by various researchers (Priestley *et al*., 1999; Ricles *et al*., 2001; Kwan and Billington, 2003; Fischer and Li, 2003). Analytical study on single bay single story RC frame with high strength reinforcement in columns also revealed less residual displacements as compared with ordinary RC frame (Qazi *et al*., 2006). Analytical results of the mixed ordinary and high strength reinforcements in columns of RC frames also showed more stable response and a delay in the formation of failure mechanisms can be achieved (Qazi *et al*., 2008). High strength reinforcements in the columns of two bays three, six and ten story frame columns also showed that more response benefits as compared with ordinary steel reinforced RC frames can be demonstrated (Qazi *et al*., 2009). Finite element model (FEM) results of two bays three story ordinary steel reinforced frame (OF) modeled on MSC.MARC and are presented here. Simulated and test results are compared. Comparison between the analytical and test results of two bays three story frames with high strength reinforcements in columns named as Passive Control RC frame (PF) is also reported here. Frames are tested and are simulated with static inverted triangular reversed cyclic loading. Both OF and PF have same geometric details. Reinforcement area ratios and material strength properties are also kept the same. In the following discussion OFT and PFT are the abbreviations used for tested OF and PF. Analytical frames are reported as OFA and PFA for ordinary and high strength steel reinforced frames respectively.

Numerical analysis and material models: Simulation of the nonlinear behavior in RC frames under seismic loads is very useful for structural safety evaluation and design. However, excessive nonlinearity and formation of failure mechanisms near collapse make it very difficult to demonstrate real behavior. Hence simplifications are needed in simulations to overcome the numerical problems. In this comparative study, a fiber model for reinforced concrete (RC) structures referred as THUFIBER (Lu *et al*., 2005) is developed, which is based on the general-purpose finite element package of MSC.MARC that carries significant ability of solving nonlinear problems. Verification of the fiber model with test results for monotonic and cyclic loadings is well documented (Lu *et al*., 2005 and Wang *et al*., 2006). In this fiber model, the concrete and the reinforcement inside the structural elements are modeled respectively with different fibers so that the cyclic behavior of material can be properly simulated. Users can define the position, area and constitutive model of each fiber. The program calculates the strain by assuming plane section remains plane and can insure that stresses on the section are in equilibrium. THUFIBER simplicity serves a useful purpose of going well into inelastic range without loosing the accuracy required. Good convergence further adds to its benefits while solving large nonlinear structural problems in MSC.MARC.

In this study for the nonlinear structural analysis with the fiber model uni-axial constitutive relations for the concrete and steel are used. For concrete, a uni-axial compressive stress-strain confinement model with a wide range of concrete strength and a linear stress strain relation for monotonic tensile loading as proposed by Legeron and Paultre (2003) shown in Fig. 1 is used.

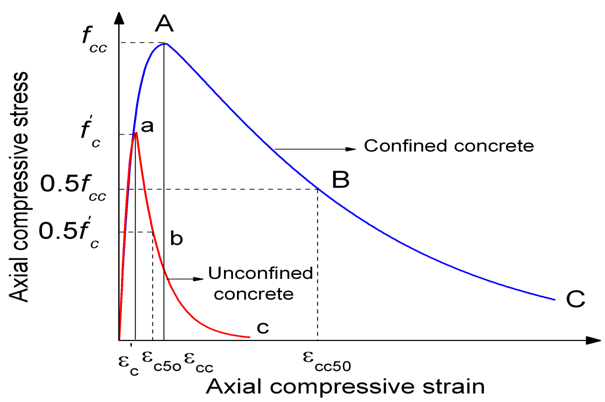


Figure 1. Uni-axial stress strain relation for confined and unconfined concrete.

The monotonic loading stress curve is assumed to form a back bone to the cyclic loading stress strain response. Unloading curve adopted here is similar to the approach presented by Mander *et al.* (1988). This model uses the crack closure function (Legeron and Paultre, 2005) which provides a stiffness recovery procedure from tension to compression and models the crack closure mechanism (Fig 2).

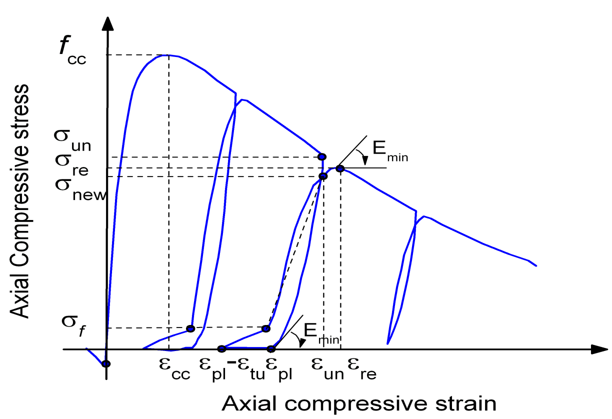


Figure 2. Hysteresis stress strain relation of concrete.

For ordinary and high strength steel a relatively simple relation for monotonic loading proposed by Esmaeily and Xiao (2005) as shown in Fig. 3(a and b) is used.

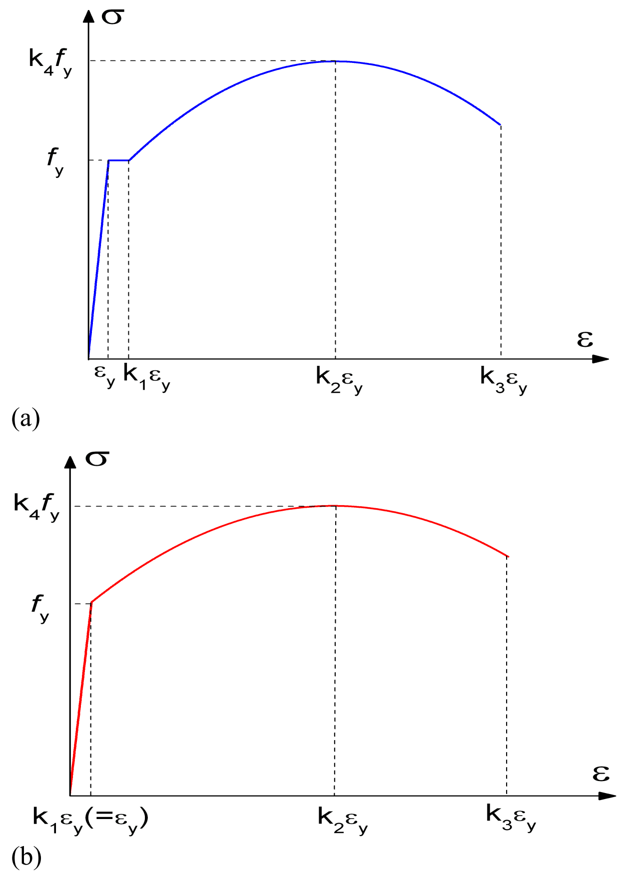


Figure 3. Monotonic stress strain relation for steel

This model with five parameters *K*1 to *K*5 is versatile and can be tuned to simulate different steel behaviors. For hysteretic behavior during cyclic loading a simplified model given by Legeron and Paultre (2003) as shown in Fig. 4 is used.

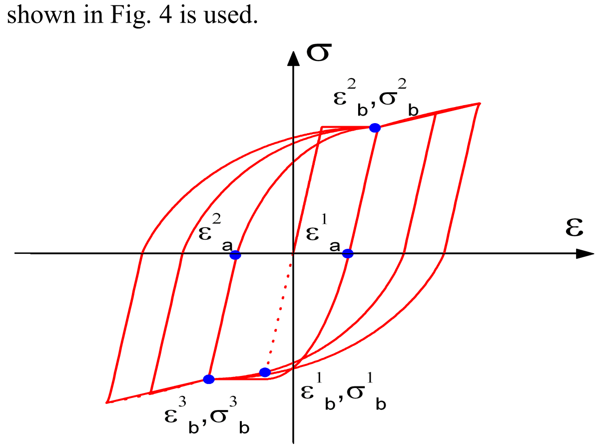


Figure 4. Hysteretic stress strain relation for steel.

For mathematical modeling of the frames beam elements are used as finite elements and section behavior is defined by using THUFIBER fiber model. Perfect bond is assumed between steel and concrete and only flexural failure is considered.

Figure 5. Experimental setup of tested frames

Comparison between analytical and test results: Material strength properties, geometry, reinforcement details and experimental setup of both the tested frames are presented in Qazi (2007). Experimental setup of the tested frames is shown in Fig. 5. To elaborate the response difference between OFT and PFT, story forces and lateral drifts are compared as shown in Fig. 6(a to c). At the second story, OFT experienced more story force and drift in the latter cycles. The residual story drift in third quadrant nearly matched for both the frames. More story force and drift are evident at the third story in OFT. Residual drift in the third quadrant is observed more in PFT. It must be realized that for OFT after some cycles of loading, excessive yielding at the columns base sections caused difficulties in maintaining triangular load pattern. Hence, a shift from load to displacement control is introduced at later stages of test which probably caused more story force in the second and the third story. However, for PFT the test was completely load control. Slightly more pinching in hysteresis of PFT as compared with OFT at large lateral drift indicates stiffness degradation. Pinching is obviously less desirable and it can be delayed in PFT by providing closely spaced lateral ties in columns.

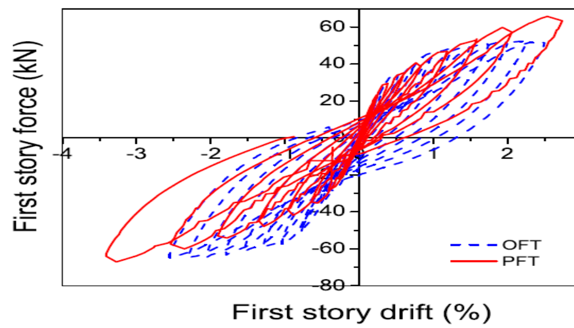
For brevity model parameters used in the numerical analysis for concrete are summarized in Table 1. Ordinary and high strength steel model parameters are given in Tables 2 and 3 respectively.

Comparison between OFA and OFT: In order to include effects of bond failures between steel and concrete steel modulus is reduced in simulation for OF. The analytical response with a proposed reduction factor of 0.5 for steel modulus is drawn in Fig. 7(a to c).

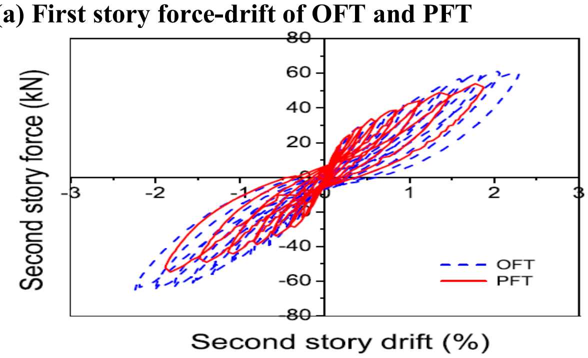
It is evident from Fig. 7a that the first story force nearly matched with test results in the first quadrant. However, residual drift obtained is of lesser magnitude in the OFA. In the third quadrant residual drift response nearly matched while story force observed is less in OFA.

Table 1: Model parameters used for simulation of concrete in OFA and PFA

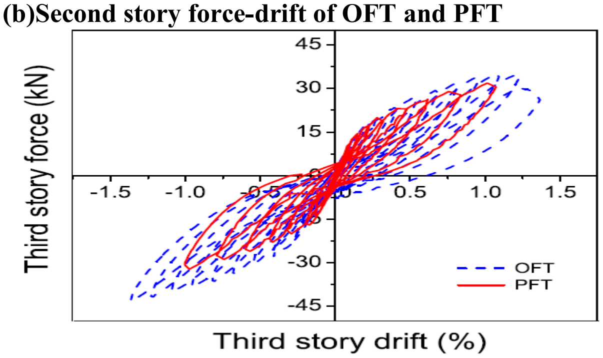
Frames	Concrete
OFA\PFA	Peak compressive strength= f'_c =21Mpa, Peak compressive strain= ε'_c =0.002, Ultimate compressive strength= 0.5 f'_c =10.5Mpa, Ultimate compressive strain= $\sigma_c 50$=0.004, Ultimate tensile strength= f_t=2MPa, Ultimate tensile strain= ε_t = 0.00015, Modulus of elasticity in tension & compression=E_c = E_t=28GPa



(a) First story force-drift of OFT and PFT



(b)Second story force-drift of OFT and PFT



(c) Third story force-drift of OFT and PFT

Figure 6 Story drift verses story force of OFT and

PFT

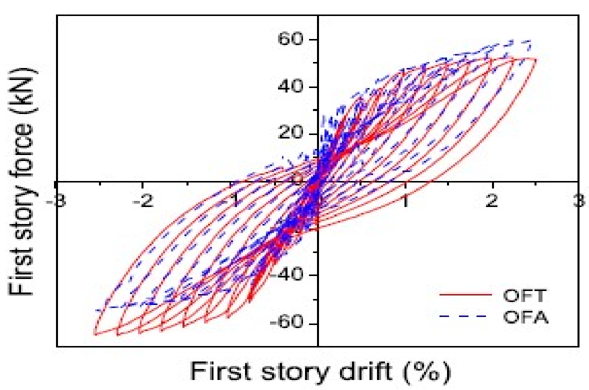
Table 2 Model parameters used for simulation of ordinary steel in OFA and PFA

Steel	Model parameters for steel in Beams\Columns in OFA & in Columns PFA
Ordinary	k_1 = 2.0 k_2 = 6.3 k_3 = 8.0 k_4 = 1.37 k_5 = 1.0 Longitudinal steel yield strength = f_y= 405MPa Modulus of elasticity = E_s=200GPa Stirrup steel yield strength= f_{yh}=367MPa

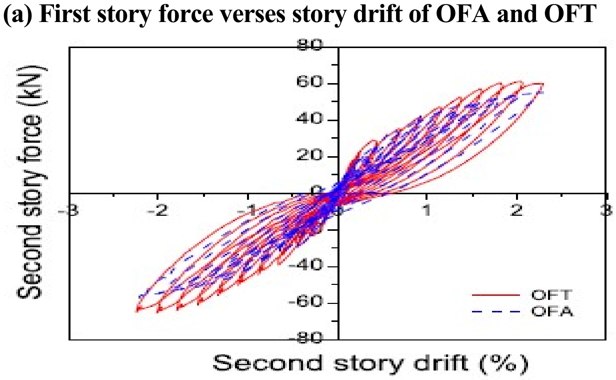
Table 3 Model parameters for simulation of high strength steel used in PFA

Steel	Model parameters for steel in Columns of PFA
High Strength	k_1=2.0 k_2 = 10.0 k_3 = 40.0 k_4 = 1.08 k_5 = 2.5 Longitudinal steel yield strength = f_y=1832.0MPa Modulus of elasticity = E_s=198GPa

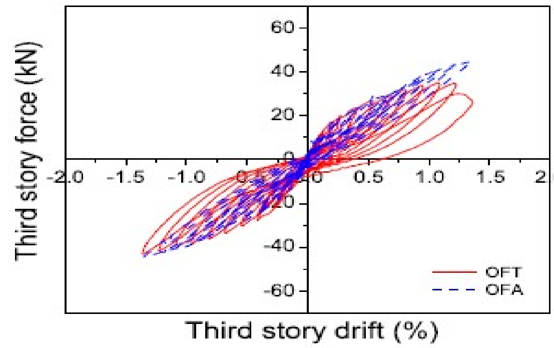
The frame story forces and drifts obtained from analysis are compared with experimental results and are plotted in Fig. (7 and 8).



(a) First story force verses story drift of OFA and OFT



(b) Second story force verses story drift of OFA and OFT



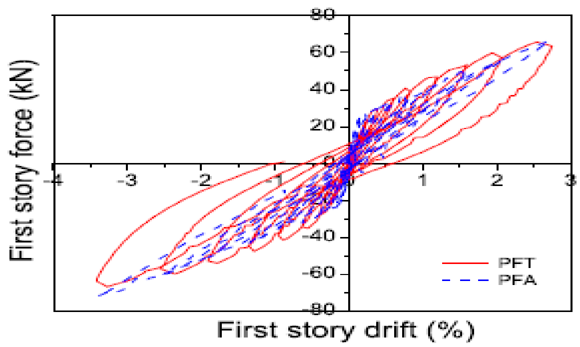
(c) Third story force verses story drift of OFA and OFT

Figure 7 Story drift verses story force of OFA and OFT

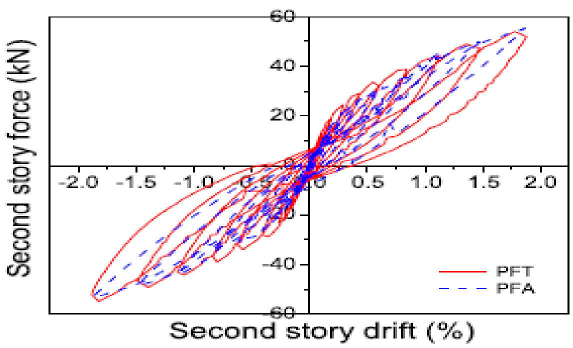
At second story analytical and test results again confirmed close similarity (Fig. 7b). In the latter cycles OFA reported a little less magnitude of story force than OFT. At the third story (Fig. 7c), in the first quadrant more story force with less residual drift is noted analytically. While, less residual drift in the third quadrant is analytically observed. Because of excessive yielding at the ground story columns a shift in control from load to displacement occurred. However in simulations it is impossible to shift the control. As fiber model hold some simplifications which may also lead to some differences in experimental and analytical results. Bond failures between steel and concrete particularly after steel yielding is more prominent in real frames and further slip at the base during test is ignored. Difference between analytical and test results can also be realized by recognizing the basic assumptions behind the formulation of fiber model.

Comparison between PFA and PFT: Fig. 8 shows a comparison between analytical and experimental response of PF. In order to incorporate drop in strength and stiffness due to bond failures and anchorage slip losses, reductions in the reinforcement modulus are proposed. Hence, in order to consider the anchorage losses in PFT, in addition to those already considered in OFT, reduction factor of 0.3 for ground story columns and 0.5 for the above story columns and beams at all floors are applied.

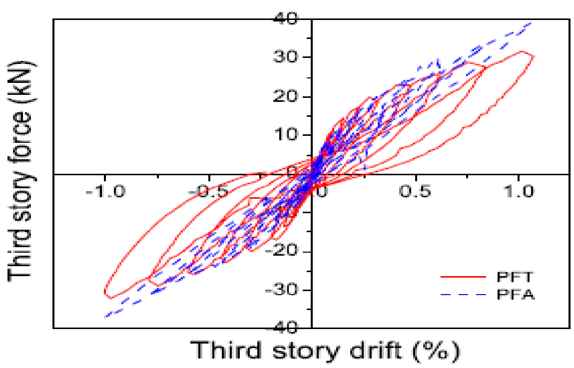
From Fig. 8a at the first story a close similarity in story force is observed. However in the unloading curves PFA showed less drift than PFT. It can be stated that proposed reduction factors are unable to exactly predict the unloading response. However at the second story test and analytical results nearly matched in story-drift response (Fig. 8b). More dissimilarity is apparent at third story (Fig. 8c). It is also noticeable that more bond slip can be expected at large drifts between high strength steel and concrete, since the concrete used is of comparatively low strength in OFT and PFT. It is also noticeable that more bond slip can be expected at large drifts between high strength steel and concrete, since the concrete used is of comparatively low strength in OFT and PFT.



(a)First story force verses story drift of PFA and PFT



(b)Second story force verses story drift of PFA and PFT



(c)Third story force verses story drift of PFA and PFT
Figure 8 Story drift verses story force of PFA and PFT

Further anchorage slip losses at different loading instants are difficult to be considered in the simulation. Moreover, inherited assumptions behind the formulation of fiber models and ignorance of shear deformations also resulted in difference between analytical and experimental response.

Conclusions: Two bays three story ordinary and high strength steel reinforced concrete frame test and analytical results are compared for evaluating difference between response mechanisms. From the comparison between test and analytical results following conclusions can be drawn;

1. Due to simplifications in fiber model some differences in experimental and analytical results are observed.
2. Bond failures between steel and concrete particularly after steel yielding is more prominent in OFT and at high steel stress in PFT.
3. Probable slip at the base during test at large lateral loads and ignorance of shear deformations also resulted in difference between analytical and experimental response.
4. Lack of control during experiment on OFT has restricted the accurate realization and good comparison with the PFT. Hence, comparison from experimental and analytical results cannot fully reveal the response benefits of the PFT. Future work is suggested by incorporating shear effects in the constitutive models. Experimental verification first at element level and then on a one bay one story frame can also be considered.

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