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RECOVERABILITY ENHANCEMENT OF REINFORCED CONCRETE BRIDGE PIERS WITH FRP COMPOSITES

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ABSTRACT

Recent developments in performance-based seismic design and assessment approaches have emphasized the importance of properly limiting the residual (irrecoverable) deformations, typically sustained by a structure after a seismic event, even when designed according to current code provisions. The aim of the study is two-fold. First, an intensive study of 39 existing bridge columns, extracted from recent research literature on the inelastic performance of FRP retrofitted columns with different deficiencies and showed no strength degradation until failure, is used to evaluate the recoverability of such retrofitted columns. The residual deformation, as a seismic performance measure, is used to evaluate the performance of these columns. Based on this evaluation, a requirement for the recoverable and irrecoverable states of FRP-RC bridges is specified. Second, to reduce the residual deformations of new RC bridge columns and to guarantee the existence of controlled-post-yield stiffness, steel bars hybridized with FRP are applied as a longitudinal reinforcement. Numerical study and experimental results showed the successfulness of using hybrid bars as a longitudinal reinforcement of concrete columns to guarantee the gradual increase of column strength and to shift its lateral deformation at the recoverable limit in comparison with column reinforced with ordinary rebars.

Introduction

Nowadays, important infrastructure is required to have not only high strength and high ductility but also usability and reparability after earthquakes. In order for structures to be able to sustain a design-level earthquake with limited or negligible damages, the rapid development of material, design, and construction techniques have been applied to improve the seismic performance and post-earthquake recoverability of reinforced concrete (RC) structures (Christopoulos 2004). Fiber reinforced polymer (FRP) materials and structural systems represent an important "new" tool in the design of protective technologies for critical structures (ACI 2003). Due to the successful performance of FRP-retrofitted columns with regard to ductility, the

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first objective of this paper is to investigate the viability of recovering FRP-retrofitted bridge columns after a moderate or strong earthquake. Due to the elastic performance and high strength of FRP, the second objective is to control the column post-yield stiffness, deformation and residual deformation in the inelastic stage through the hybridization of the main steel reinforcement bars with FRP.

Idealized load-deformation model of FRP-RC damage-controllable structures

The need for structural systems to withstand large earthquake forces without compromising life-safety and the recent progress of experimental and analytical studies on retrofitting of deficient bridges have brought the challenge of designing a quickly recoverable structure to the research forefront. Here, the authors propose a mechanical model for a damage-controllable structure using FRP. Fig. 1 shows the mechanical model of the proposed structure, where the lateral response proceeds along O-A-B-C-D-E-F. The behavior of a general RC flexural structure whose lateral response is along O-A′-B′-C′-D′-F′ is also given for comparison. Prior to the cracking of concrete, lines OA and OA′ corresponding to both types of structures share similar stiffnesses. The stiffness of the proposed structure, K_1, is slightly greater than K_1' of the general RC structure after concrete cracking. The most remarkable difference occurs after the yielding of the steel reinforcement: after point C and C′. For the general RC structure, the deformation increases dramatically almost without any increase in load carrying capability: along line C′D′ no post-yield stiffness is demonstrated. However, with the proposed approach, the structure can still carry the load even after the steel reinforcement yields and hardening behavior has been exhibited along line CD. The stiffness K_2 between points C and D is termed the "secondary stiffness" in this paper.

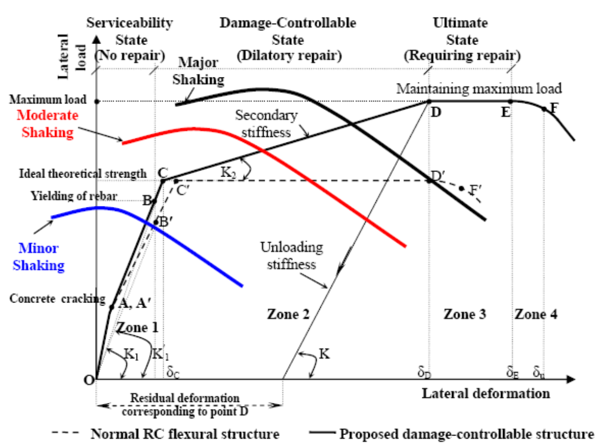

Figure 1. Idealized load-deformation behavior of proposed damage controlled structures

Due to the existence of secondary stiffness, the dramatic increase in deformation and residual deformation can be effectively controlled after the reinforcement yields, and the load carrying capacity can be further improved. Based on the codes requirements for ductile structures to withstand strong earthquakes, the proposed structure is characterized by the part DE after the hardening zone, where favorable ductility is demonstrated. The ultimate drift (δ_u) corresponding to point F or F′ for the proposed structure and the general RC structure, respectively, is defined for both structures to be at 20 percent strength decay. The use of 20 percent strength decay as the failure criterion is consistent with that employed by previous researchers, since it is reasonable to accept some strength decay during seismic response of a structure before it can be considered to have failed.

According to the mechanical behavior shown in Fig. 1, the load-deformation of the proposed structure can be divided into four main zones; Zone 1: from point O to B; Zone 2: from point B to D; Zone 3: from point D to E; and Zone 4: after point E. Zone 1 corresponds to a stage of no damage or concrete cracking. Under a small earthquake, the mechanical behavior should be controlled in this zone, and the original function of the structure can be maintained without any repair and displacement of elements. Zone 2 corresponds to the hardening behavior after the yielding of steel reinforcements, where a distinct secondary stiffness is demonstrated and the dramatic deformation can be effectively controlled. Under a medium or strong earthquake, the mechanical behavior of the proposed structure should be within zone 2. Thus, damage can be effectively controlled by the secondary stiffness. The original function of the structures can be quickly recovered through repairs after a medium or large earthquake. Zone 3 corresponds to ductile behavior after hardening, where favorable ductility is demonstrated under a large earthquake. The proposed structure can be kept in place for a relatively long time without collapse during a large earthquake, though severe damage may occur. The original function of the structures may be recovered through the replacement of some elements. During a severe earthquake, the mechanical behavior may enter zone 4 with collapse.

The proposed mechanical model can satisfy a seismic design philosophy that holds that the structure suffers no damage under a small earthquake, exhibits prompt recoverability under medium earthquake, and does not collapse under a large earthquake.

Post-yield stiffness and residual deformation in seismic design

Since the strength requirement for strong earthquake (L2) is much higher than the frequent one (L1), the existing RC bridge columns that satisfy L1 must be enlarged and/or increased in reinforcement to meet the new requirements. However, if a suitable FRP retrofitting is used effectively to assure the post-yield stiffness, the L2 design criteria may be met without dramatically increasing the column section size or the amount of reinforcement. Moreover, minimum irrecoverable deformation would be in case the structure has positive post-yield stiffness. Fig. 2 shows three potential responses of a structure under the action of an earthquake. The key difference between these responses is the inelastic performance, i.e., negative, zero, or positive post-yield stiffness. At the same lateral drift, unloading stiffnesses are parallel in accordance with Takeda model (Takeda *et al*. 1970), where, the unloading stiffness K is a function of column initial stiffness K_1 and ductility μ, (Eq. 1). It is evident from Fig. 2 that negative post yield stiffness results in a large residual displacement response, which in turn is a disadvantage that should be avoided to quickly recover the structure.

$$K = \frac{K_1}{\sqrt{\mu}} \tag{1}$$

The 1996 Seismic Design Specifications of Highway Bridges in Japan specifies that the residual displacement should not be greater than 1% of the piers height (JSCE 2000), and it provides the following equation for evaluation (Eq. 2):

$$\delta_R = C_R(\mu_R - 1)(1 - r)\delta_y \tag{2}$$

where: δ_R = residual displacement of a pier after earthquake, μ_R = response ductility factor of pier, $r = (K_2/K_1)$ bilinear factor defined as a ratio between K_2 (post-yield stiffness) and K_1, C_R factor depending on the bilinear factor r, and δ_y = yield displacement.

The equation explicitly verifies that as the r ratio increases, the residual displacement will decrease accordingly, and it can be concluded that the piers with high r values have a higher seismic performance.

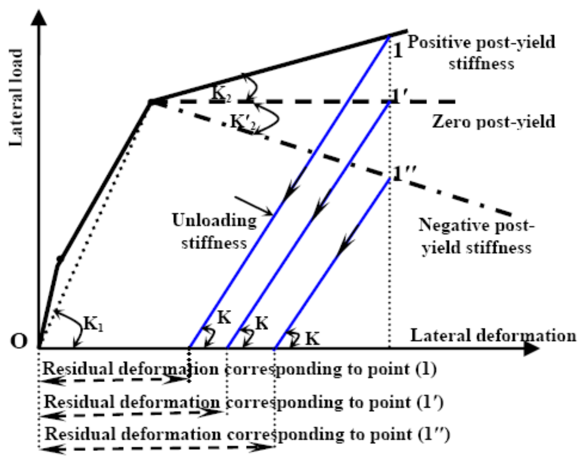

Figure 2. Effect of post-yield stiffness on the residual deformation

Existing RC Bridge Columns

Seismic Behavior of Existing RC Bridge Columns

Lap splice failure at the connection between the footing and the column, shear failure, and confinement failure of the flexural plastic hinge region are the failure modes observed in existing reinforced concrete bridge columns under a seismic load/deformation input (Seible *et al.* 1997). These failure modes are potentially related to poor detail in the longitudinal lap splices, improper transverse confinement, and insufficient shear strength.

Realizing post-yield stiffness of existing FRP-retrofitted RC bridge columns

Fahmy *et al.* examined the ability of retrofitting existing RC bridge columns using fibre-reinforced plastic (FRP) jacketing to guarantee a quick recovery of the original functions after a moderate to strong earthquake attack. An up-to-date literature search pertaining to the performance of retrofitted columns with FRP was performed. The inelastic performance of 109 retrofitted columns with lap-splice deficiency, flexural deficiency, or shear deficiency was determined from the literature review. From the envelope curve of the hysteretic response of retrofitted and repaired columns, sixty-one columns exhibited that the idealized lateral load-displacement relation has the stable post-yield stiffness.

Performance evaluation of FRP retrofitted columns using residual deformation index

To categorize columns, successfully achieved post-yield stiffness, in accordance with the required recoverability after an earthquake, it is necessary to consider the residual deformation as an important performance index. So, the residual deformation corresponding to the endpoint of the post-yield stiffness is experimentally defined from the hysteretic curves of 39 scale-model tests available from the database (Fahmy *et al.*, 2009).

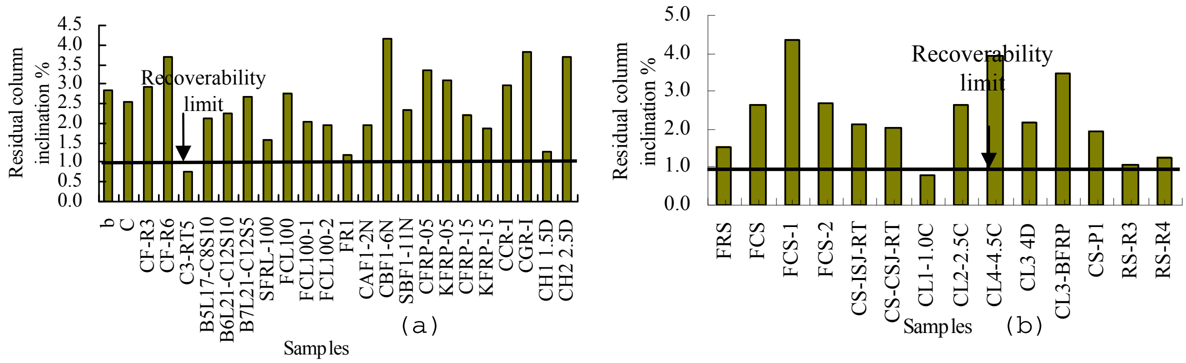

Figure 3. Residual column inclination corresponding to the endpoint of post-yield stiffness for columns with (a) flexural or lap-splice deficiencies; and (b) shear deficiencies (Fahmy *et al.* 2009)

This residual deformation, which is defined as the displacement of zero-crossing at unloading on the hysteresis loop from the end point of post-yield stiffness, should not exceed 1% of the column height for rapid restoration of structural functions after an earthquake. Residual column inclination, defined as the ratio of residual deformation corresponding to the endpoint of post-yield stiffness to column height, for columns with flexural or lap-splice deficiencies is shown in Fig. 3 (a). Fig. 3 (b) represents columns with shear deficiencies. Weakly confined circular columns (CH1-1.5D, CL1-1C) and rectangular columns (RS-R3, RS-R4, FR1, FRS, and Specimen No. 3) have residual drift ratios fluctuating around the recoverability limit. On the other hand, the residual inclination of the remaining columns, whether originally suffering shear, flexural, or lap-splice deficiencies, is in excess of the recoverability limit. This is an indication that the control of the irrecoverable deformations of existing structures using FRP composites is a future challenge toward achieving the aim of ductile recoverable structures. A longer treatment for this aspect is studied by Fahmy *et al.* (2009).

Definition of the Limit States of FRP-RC Bridge Columns

Measuring the seismic performance of the FRP-retrofitted columns based on post-yield stiffness and residual deformation shows that many columns successfully achieved the secondary stiffness; however, the residual deformation corresponding to the endpoint of the secondary stiffness is in excess of 1.0%, as shown in Figures 3 (a and b). Based on these results, the limit state for recoverable columns will not be the endpoint of the post-yield stiffness, and a redefinition of the endpoint of the recoverable state using the nonlinear pushover test results becomes necessary. The relationship between the column drift ratios and the corresponding column residual drift ratios at the column theoretical strength, recoverable limit, and maximum strength of these columns are plotted in Fig. 4. It is clear that a residual drift ratio of 1% does not correspond to a specific column drift ratio, since many parameters affect the performance of these columns. However, it is interesting to stress that there is a zone of the drift ratio between 2 and 3.5 % within which the recoverability limit state could be checked. Hence, the endpoint of the recoverable state can be defined by evaluating the residual inclination value within this zone.

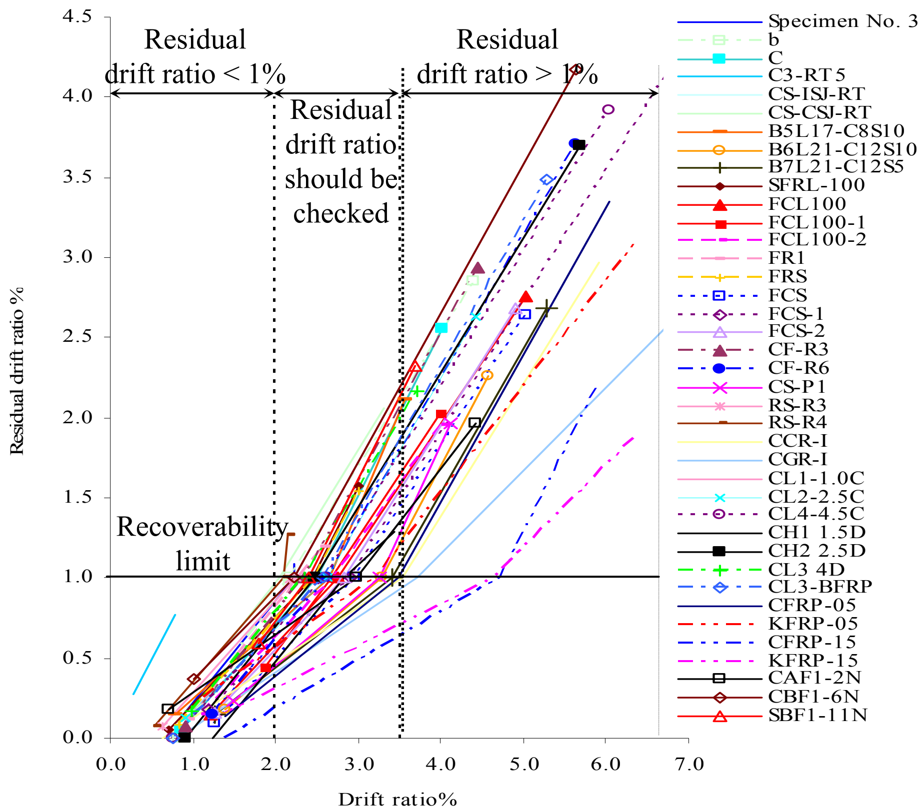

Figure 4. Column drift ratio versus residual drift ratio

In conclusion, Fahmy *et al.* 2009 recommend three limit states for FRP-RC bridge columns as shown in Fig. 5. The first state is the state of pure recoverability whose end corresponds to column drift ratio 2% as shown Fig. 5. Here, the residual deformation of all the represented columns is below the recoverability limit. The second state is the state of recoverability limit check which falls between 2% and 3.5% column drift ratios. The third one is the irrecoverable state, where the residual deformations exceed the recoverability limit.

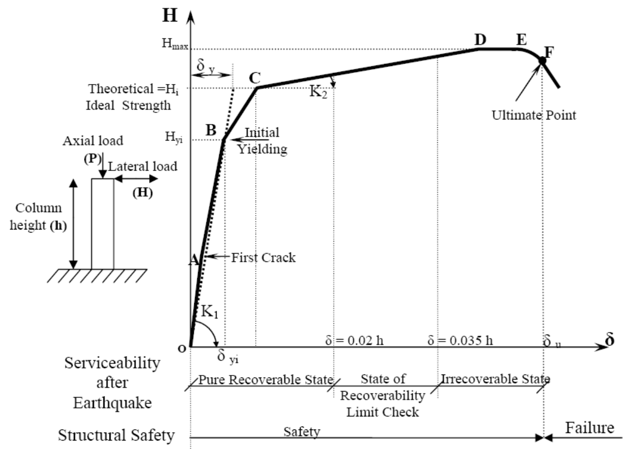

Figure 5. Recoverable and irrecoverable states (Fahmy *et al*. 2009)

New RC Bridge Columns

Concept of Steel-FRP Composite Bar (SFCB)

The novel concept here is that hybridization of fibers with steel bar could enhance the steel inelastic response through strain hardening by the end of the elastic stage, and this strain hardening region could be controlled depending on the type and amount of FRP. Also, the permanent deformations would be reduced due to the elastic performance of FRP, which is characterized by approximately zero residual deformations. Moreover, FRP has a high resistance against corrosion. Consequently, hybridizing steel bars with fibers is a reasonable solution to control the structure performance and enhance its serviceability (Wu 2006). Static tensile tests were carried out to find the mechanical properties of the new hybridized steel rod with different amounts of carbon FRP (CFRP) and Basalt FRP (BFRP). Factory products of SFCB can be seen in Fig. 6 (a) (Wu 2009).

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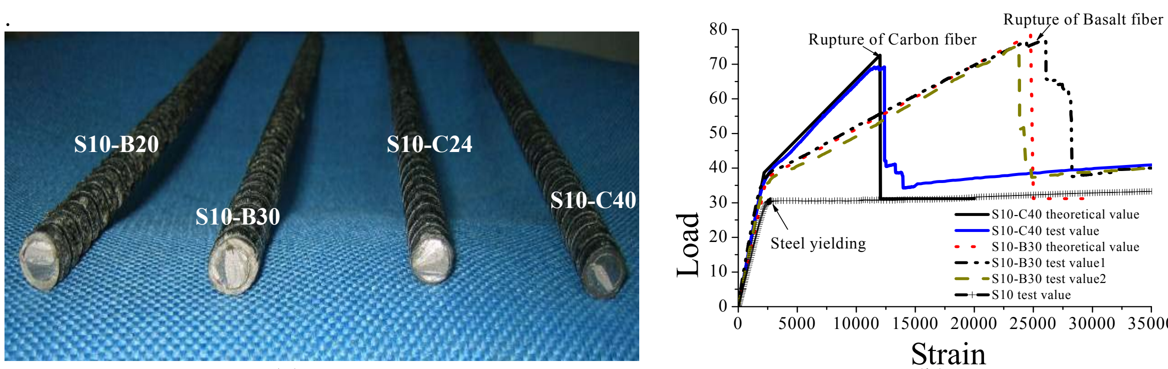

(a) (b)

Figure 6. (a) factory products of SFCB (b) Load-strain relationship of ordinary steel bar, SCFCB, and SBFCB (Wu 2009)

Fig. 6 (b) shows that the post-yield stiffness of steel bar could be controlled after hybridization with FRP, where FRP amount controls the definition of the slope value of the fulfilled post-yield stiffness and FRP type defines the end point of this stiffness. Test results revealed that the elongation rate of the fiber used is responsible about definition of end point of the achieved post-yield stiffness, where steel carbon-fiber composite bar (SCFCB) and steel basalt-fiber composite bar (SBFCB) realized different axial strains by the end of the achieved post-yield stiffness: SBFCBs have stable post-yield stiffness till a strain value almost double of that of SCFCBs. Based on statistic analysis of the cyclic tensile test data (Wu *et al.*), the equation for the unloading stiffness of SFCB is

$$E_u = \frac{E_I}{(1+\gamma\dfrac{\varepsilon_p}{\varepsilon_y})} \tag{3}$$

where γ ($\gamma \geq 0$) is the degradation coefficient of stiffness (for ordinary elastoplastic steel bar $\gamma = 0$ and for SFCB with stable post-yield stiffness $\gamma > 0$; by regression of test data when E_{II}/E_I, post-yield stiffness ratio is between 0.097 and 0.196, γ is 0.055, R^2=0.9488). This equation mainly takes into account the effects of the plastic development after the yielding of SFCB on its unloading stiffness.

A computer program (Mazzoni 2009) depending on fiber analysis is used for the analytical study of columns performance reinforced with SFCBs and ordinary rebars. Due to the importance of the effect of bond between the new bars and concrete on the column performance, a zero-length section element (Zhao 2007) which reflects the effect of strain penetration of longitudinal bars into bridge footing was considered in this study. Figures 7 (a, b & c) show comparison between the calculated reversed cyclic performances and the experimental results of three columns, where column (CS14) is reinforced with the ordinary steel (twelve 14-mm-diameter), and the other two samples were reinforced with twelve S10-B30 and S10-C40, respectively.

The tested CS10-B30 and CS10-C40 showed a close similarity in the relation between column drifts and the corresponding lateral load till a lateral drift of 15-mm, at which light rupture sound of the carbon fibers of column CS10-C40 was noticed, and at 25-mm louder sound of the ruptured fiber was continuous till the end of the test, Fig. 7 (c). Column CS10-B30 was still carrying load till a lateral deformation of about 30-mm, and then louder sounds at 35-mm lateral drift were due to the continuous rupture of basalt fibers, Fig. 7 (b). Deformation of column CS14 increased without any increase in load carrying capability, and strength degradation started at lateral displacement of 35-mm due to buckling of longitudinal steel bars at the plastic hinge zone, Fig. 7 (a).

It is clear from these figures that the main difference could be noticed form the inelastic stage after steel yielding, where column reinforced with SFCBs showed a clear manifestation of the post-yield stiffness, meanwhile the strength of column S14 kept constant during the inelastic stage. Furthermore, Figs. 7 (d & e) reflect the possibility of shifting the column lateral drift, corresponding to the recoverability limit, to a higher value after steel bars with FRP.

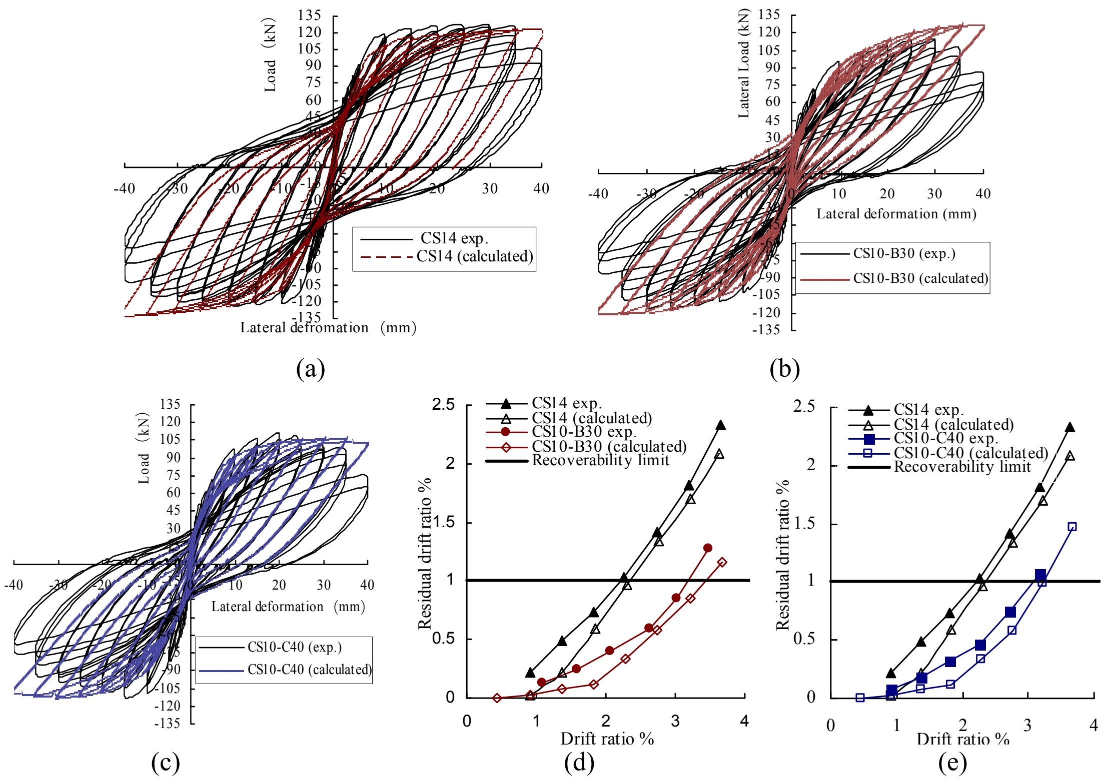

Figure 7. Comparison between analytical and experimental results of column (a) CS14, (b) CS10-B30, (c) CS10-C40, and calculated and experimental residual drift of the columns (d) CS14 and CS10-B30, (e) CS14 and CS10-C40

Conclusions

This paper discussed the seismic performance of the FRP-RC structures from the perspective of the required recoverable performance after a moderate to strong earthquake. The following conclusions are derived from the inclusive data.

1) For enhancement of urban safety, the important structures are required to have not only high strength and ductility, but also the required recoverability. Moreover, to ensure the required structural recoverability, a sufficient controllability is also necessary. 2) In comparison of FRP-RC structures with general RC structures, an integrated property of FRP-RC structures is realised, including initial and secondary stiffness, load carrying capacity, and ductility. 3) The FRP-RC structures (existing deficient columns with FRP jacketing, and structures reinforced with SFCBs) can realise the seismic design goal of no damage during small earthquake, prompt recoverability during a medium or strong earthquake, and no collapse during a huge earthquake. 4) Some structural recoverability can be realised through FRP strengthening. However, it needs to be enhanced through a FRP design. 5) An advanced strengthening design methodology and design guideline of considering and evaluating structural recoverability is desirable. For this, structural recoverability for different existing and strengthened structures should be deeply understood through future study efforts.

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