



Seismic Behavior and Assessment of Concrete Walls Reinforced by CFRP Bars

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Doctoral Dissertation

Seismic Behavior and Assessment of Concrete
Walls Reinforced by CFRP Bars

CFRP 筋補強コンクリート壁の耐震性能と
評価方法

July 2021

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CHAPTER ONE

1. Introduction

1.1 Backgrounds

More than thirty-five thousand M 5+ earthquakes had occurred, which took lives of about eight hundred thousand people, during last two decades (2000-2019). Magnitude 7 and greater earthquakes exceeded 300, and some even reached M 9+ such as Indian Ocean Earthquake (M 9.1, 2004) and Tohoku Earthquake (M 9.1, 2011) [1.1]. These mega-earthquakes caused a massive loss of live and property.

It has always been an important issue that how to resist terrible earthquakes. Structures that possess resisting capacity to avoid collapse are undoubtedly desirable, which can save lives from the disastrous earthquakes. Lots of researchers have devoted themselves to aseismic structures for the purpose of protecting people against earthquake hazards. While earthquakes cause lateral and vertical inertia forces to structures, the lateral inertia force has been generally noticed because it is the primary cause for the damage of structures. So the lateral force resisting members, such as columns and walls, are main earthquake resisting components of structures. Walls are usually utilized in high-rise structures because of their large lateral stiffness.

Ductile reinforced concrete walls with characteristics of adequate ductility and large energy dissipation capacity can sustain sufficient lateral deformation and dissipate earthquake energy through large plastic deformation, which makes ductile reinforced concrete walls avoid collapse and achieve the goal of limiting casualties when hit by mega-earthquakes. So ductile reinforce concrete walls have been widely used in high-rise

buildings as lateral force resisting components. However, large plastic deformation accounts for deformation and energy dissipation capacities, and may cause too large residual deformation and damage to concrete walls, as shown in Fig. 1-1.



Fig. 1-1 Damaged ductile concrete walls during Chile earthquake [1.2]

It is very difficult to repair these ductile concrete walls with large residual deformation after earthquakes, and the whole building has to be demolished for reestablishment resulting in huge consumption of time and expense. On the other hand, Lifeline engineering structures, such as power station, hospital etc., should recover function rapidly for conduct of relief effort. Therefore, resilience of structural components inclusive of concrete walls, which means little residual deformation also deserves attention besides sufficient resistance to strong earthquakes.

Resilience has recently gained attention in the earthquake engineering community, just as the theme of 17th Word Conference on Earthquake Engineering says, “Toward Resilient Society” [1.3]. Resilient structures are engineering structures raised from the viewpoint of resisting earthquakes and recovering structure function immediately with simple repair or without repairing even when they are hit by mega-earthquakes. Many resilient structures, for example, rocking structures, self-centering structures, repairable structures, have been proposed [1.4-1.5].

The resilient concrete wall is a kind of components of resilient structures that can not only exhibit well lateral resistance avoiding collapse during earthquake but also limit residual deformation, making rapid recoverability possible. Typical lateral force-drift (V-R) relationships of resilient and ductile concrete walls are compared in Fig 1-2. As obvious from Fig. 1-2, the resilient concrete wall can maintain stable lateral capacity up to large drift and limit residual deformation at a low level. There have been many studies on resilient concrete walls conducted by researchers because of excellent seismic behavior

mentioned. These researches will be reviewed in the next section to find out remaining problems which will be research contents of this dissertation.

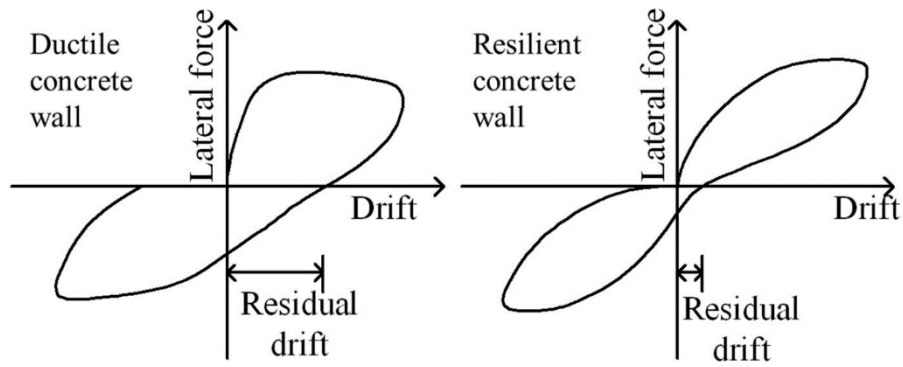


Fig. 1-2 Lateral force-drift (V-R) relationships of ductile and resilient concrete walls

1.2 Studies on resilient concrete walls

Resilient concrete walls should reach two goals, sufficient resisting capacity to withstand mega-earthquakes and resilience to recover function of buildings after disasters. One of the methods to attain these capacities is to use the unbounded pre-stressing reinforcement. Priestley conducted an experiment of a 60% scale five-story precast concrete building consisting of unbonded post-tensioned precast (UPTP) concrete walls in one direction, and precast frames in the orthogonal direction under pseudo-dynamic loading in 1999. The UPTP concrete walls comprised four precast panels that vertically connected to each other and to foundation by unbonded vertical post-tensioning threaded bars. Experimental results showed that there was a residual drift of 0.06% after sustaining a peak drift of 1.8%. Besides, damage was also minimal in the wall direction, which revealed well resilience of UPTP concrete walls under seismic loads [1.6].

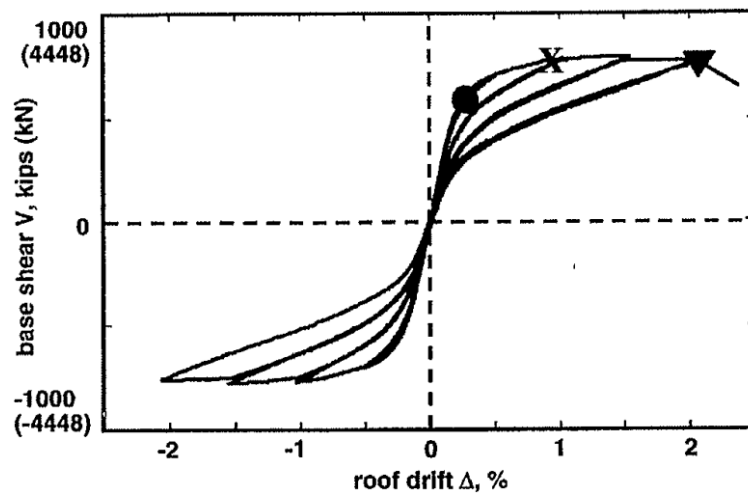


Fig. 1-3 Hysteretic behavior of UPTP wall [1.7]

Kurama investigated seismic behavior of UPTP concrete walls using an analytical model. Apart from unbonded post-tensioned (PT) steel bars or multi-strand tendons, spiral reinforcing steel was also used in wall panels near the base of the wall to confine concrete. UPTP concrete walls under cyclic load exhibited the nonlinear-elastic behavior, as shown in Fig. 1-3, which means that the walls return back towards original position with little residual displacement after earthquakes. Behavior under earthquake load were also investigated through more than 200 nonlinear dynamic time-history analyses by Kurama.

The response of UPTP walls oscillated around the zero-drift position showing significant self-centering capability too [1.7-1.9]. To reduce the lateral drift response and compensate for the low energy dissipation capacity of UPTP walls, Kurama proposed a supplemental passive energy dissipation system using linear viscous fluid dampers, introduced a design approach for the supplemental energy dissipation system and analytically verified effectiveness of the proposed dissipation system and the design approach [1.10]. Besides, Kurama found that the use of bonded mild steel reinforcement can also result in a considered reduction in lateral displacements and a significant increment in the energy dissipation of UPTP walls [1.11, 1.12].

As normal steel bars and multi-strand tendons, carbon fiber reinforced tendons could also offer resilience to UPTP concrete walls, which was verified by Holden and Restrepo [1.13]. They constructed and tested a UPTP concrete wall incorporated post-tensioned unbonded carbon fiber tendons and steel fiber reinforced concrete and a conventional precast reinforced wall as reference specimen. The UPTP concrete wall had virtually no residual drifts after experiencing a peak displacement well in excess of 3% drift, whereas the conventional precast reinforced wall left 2% residual drift unloading from the same peak drift level [1.13].

Restrepo and Rahman experimentally investigated seismic behavior of the UPTP walls with longitudinal mild steel reinforcement as energy dissipators and verified that the walls incorporating energy dissipators displayed a so-called “flag-shaped” hysteretic response and had equivalent viscous damping ratios of up to 14% [1.14]. Dang and Lu [1.15] used finite element methods to analyze seismic performance of the walls fabricated and tested by Restrepo and Rahman [1.14]. The analytical results indicated that the proposed simulation methods could capture the hysteresis behavior, the stress of unbounded pre-stressed tendon and the stress/strain parameter of the walls [1.15]. Besides, Dang and Lu also conducted experimental and analytical investigation on UPTP walls with horizontal bottom slit. According to their experimental results, placing horizontal slits symmetrically at the wall ends on the wall-foundation interface of UPTP walls could obtain better energy dissipating ability than the traditional UPTP walls [1.16, 1.17].

Perez conducted experimental and analytical research on UPTP concrete walls with

parameters of total area of PT steel and their locations, initial stress in PT bars, initial stress in concrete due to posttensioning and confining reinforcement details in the base panel and founded that the analytical model was very accurate for walls with higher prestress, but overestimated the base shear capacity of walls with lower prestress [1.18-1.20], which indicated that the prestressing level has a significant effect on cyclic behavior of UPTP walls and need to be investigated in detail.

Besides the performance of unbonded reinforcement, end anchorages also deserve necessary attention. Belleri discussed the dynamic response of a half-scale three-story precast concrete building with the lateral force resistant system of UPTP walls subjected to extensive shake-table testing. The UPTP walls exhibited satisfactory seismic resistance with limited flaking of concrete at wall toe under design basis earthquake. However, the complete tendon failure with uncontrolled wall rocking and unacceptable drift demands were observed during the subsequent maximum considered earthquake shaking test, which showed that reliable end anchorage of post-tensioned tendon and post-earthquake inspection of the tendon condition were essential for UPTP concrete walls [1.21].

As reviewed above, although UPTP walls can reach seismic resisting goals of resistance and resilience, complexity of construction, loss of prestress and failure of anchorage are inevitable problems to be considered in engineering application. To achieve a more convenient method realizing resilience of concrete walls, Sun et al. proposed a simple reinforcing method using weakly bonded ultra-high strength (WBUS) longitudinal bars without prestressing. One of the WBUS rebars is SBPDN rebar that has a specified yield strength of 1275 MPa and a low bond strength of about 3 MPa [1.22]. Placing SBPDN rebars as lumped longitudinal reinforcement can provide cantilever concrete walls with drift-hardening lateral resistance up to 3% drift with little residual drift under cyclic loading test (see Fig. 1-4) [1.23]. Fujitani and Sun also investigated seismic performance of SBPDN reinforcing concrete walls under pseudo-dynamic load and found that equivalent lateral resistance and smaller residual drift were obtained compared with walls under static cyclic lateral loading test [1.24]. Besides, Sun et al. found that the use of SBPDN rebars as diagonal and triangular reinforcement can also provide drift-hardening capacity and resilience for concrete walls [1.25-1.27].

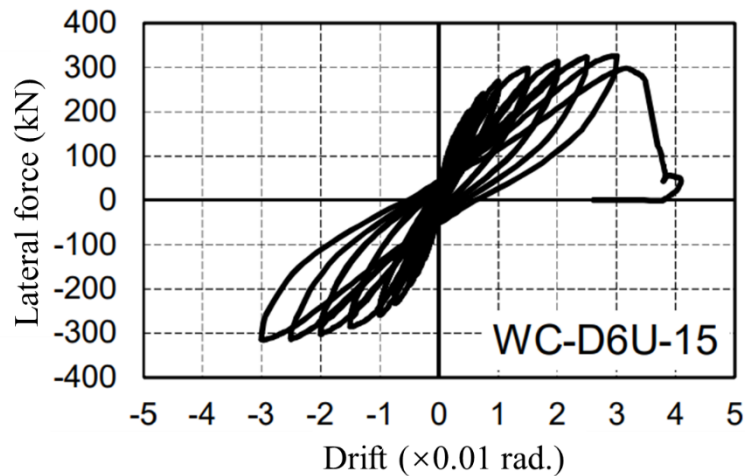


Fig. 1-4 Hysteretic behavior of SBPDN reinforcing wall [1.23]

Sun and his colleagues have not only conducted experimental studies on concrete walls reinforced by SBPDN rebars but also done analytical research. Because of the ultra-high yield strength and the low bond strength, slippage of SBPDN rebars in concrete is unavoidable, and traditional analytical methods to reinforced concrete components based on the plane-remain-plane assumption cannot be applied to the design or performance evaluation of this new resilient concrete walls. A new analytical method considering effects of slippage of ultra-high strength rebars was first proposed by sun et al. to assess seismic behavior of concrete columns reinforced by ultra-high strength deformed rebars using so-called finite spring method [1.28-1.32]. That method was modified according to subsequent studies on the bond-slip relationship of SBPDN rebars in concrete and used to predict hysteretic behaviors of concrete columns reinforced by SBPDN rebars [1.22, 1.33-1.34]. The applicability and accuracy of the proposed analytical method used in assessing SBPDN rebars reinforcing concrete walls were also verified [1.23, 1.24].

Yuan et al. have also proposed another WBUS rebar, PC strands, which have higher yield strength and almost identical bond strength compared with SBPDN rebars. Fig. 1-5 shows hysteretic behavior of concrete wall reinforced by the PC strands. As shown in Fig. 1-5, the experimental results by Yuan have indicated that concrete walls reinforced by PC strands could exhibit drift-hardening capacity up to 2.5% drift with little residual drift of 0.41% after unloading from 2.0% drift, while the residual drift of reference ductile RC walls reached 1.08% after unloading from the same drift level [1.35]. Seismic

performance of concrete walls reinforced by PC strands under relatively high axial load ratio are also experimentally investigated by Yuan et al. PC strands reinforcing walls with sound confinement in their boundary elements could reached 2% drift with residual ratio not exceeding 0.35% and no degradation of lateral resistance [1.36].

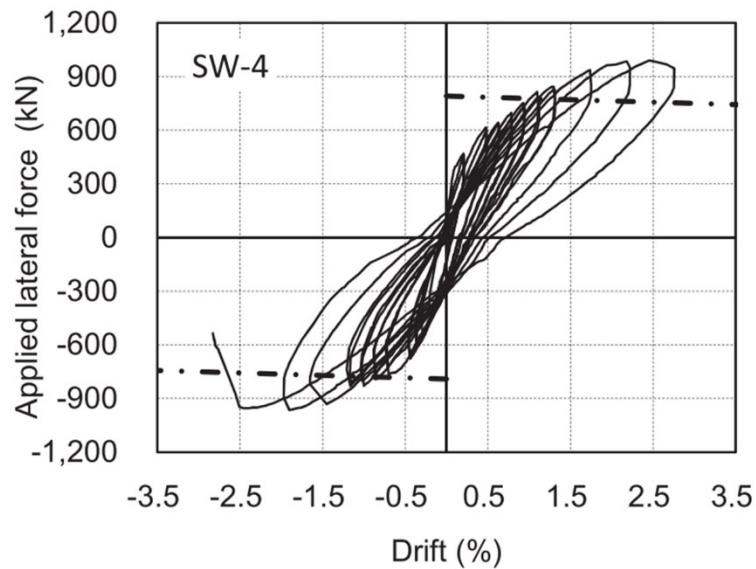


Fig. 1-5 Hysteretic behavior of PC strand reinforcing wall [1.35]

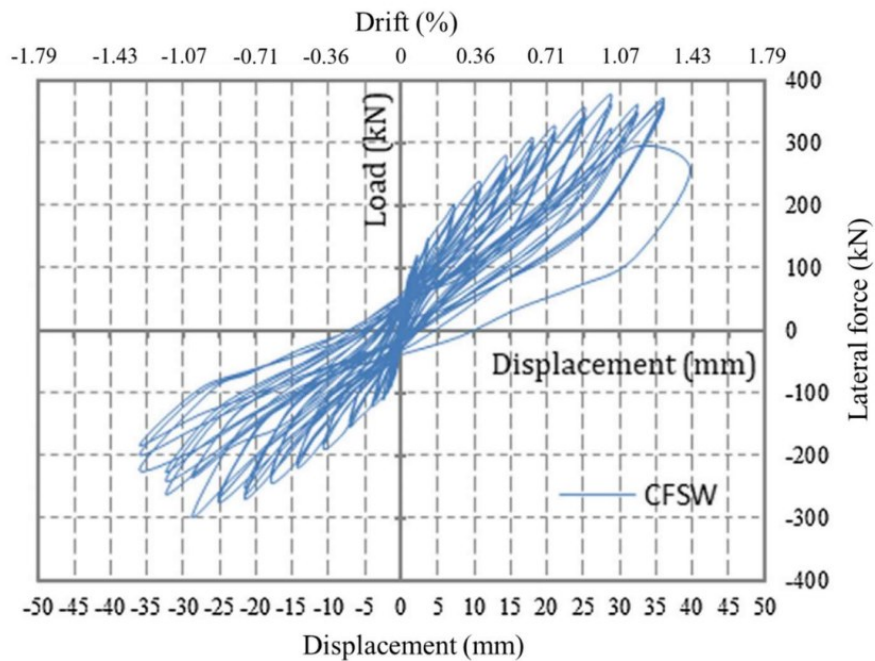


Fig. 1-6 Hysteretic behavior of CFRP reinforcing wall [1.39]

As mentioned above, SBPDN rebars and PC strands have characteristics of ultra-high yield strength and low bond strength as WBUS rebars, which can provide resistance and resilience with no need for prestressing. CFRP bars also have the similar features and have been widely used in engineering structures [1.37]. The validity of resilience provided by CFRP bars were verified by Zhao et al (see Fig. 1-6), whose experimental investigation indicated that concrete walls reinforced by CFRP bars had smaller maximum crack widths, residual crack widths and residual deformations with decrease of 60%, 70% and 90% than ordinary RC walls respectively [1.38,1.39].

1.3 Studies on concrete confined by transverse reinforcement

The well confined concrete with sufficient deformability is another essential element for resilient concrete structures. Studies on confined concrete could trace back to 1928 when Richart conducted investigation of concrete cylinders with active confinement from lateral fluid pressure and found that the lateral stress provided by fluid pressure could enlarge compression stress and corresponding strain of concrete. Besides, relationships between confined concrete strength and corresponding strain with lateral stress were quantitatively analyzed [1.40]. In 1929, Richart also found the approximately same enhancement of passive confinement pressure by using spiral steel reinforcements that have been widely utilized in RC components, especially the earthquake-resisting columns for ductility [1.41].

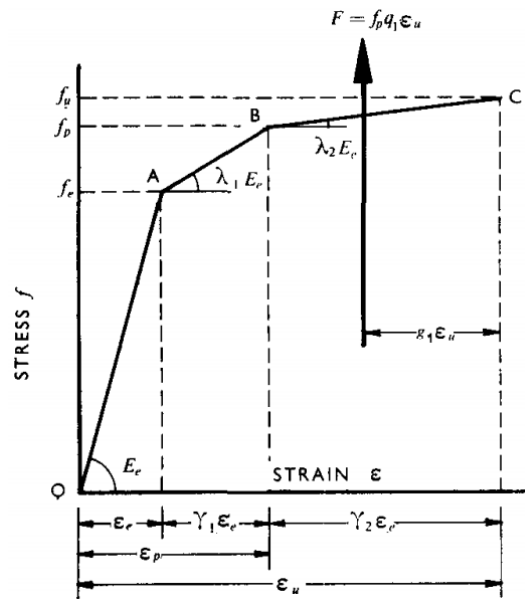


Fig. 1-7 Stress-strain curves for confined concrete proposed by Chan [1.42]

The complete stress-strain curve model for concrete confined by transverse reinforcement is indispensable in analyzing overall performance of reinforced concrete components, and numerous researchers have done many works to accurately predict properties of hoop-confined concrete [1.42-1.63]. Chan proposed a trilinear stress-strain relationship of hoop-confined concrete predicting only the ascending part, as shown in Fig. 1-7. This model can be used in not only circular but also rectangular hoop-confined concrete and

experimental results indicated that circular hoop-confinement was distinctly more efficient than rectangular confinement. It should be noted that only the volumetric ratio of lateral steel to concrete core was considered as variable in this model [1.42].

A stress-strain relation for the concrete confined by rectangular transverse reinforcement was proposed by Soliman and Yu [1.43]. Their model can cover variables such as the spacing and area of hoops, the ratio of the confined area to the gross area, the shape of the concrete cross-section. This model could predict the descending part of the stress-strain relation till a 20% drop in the maximum stress. Soliman and Yu also drew conclusions that increasing the spacing of the hoops decreased considerably the confining effect, which even was weakened to 0 when the spacing is more than 250 mm and that the increase of confinement by increasing the area of hoops was quite small compared with that by decreasing the spacing of hoops [1.43]. It can be presumed from their investigation that the confining effect by hoops small in diameter with close spacing may be higher than that by thick hoops with large spacing when the volumetric ratio is same for both cases.

Kent and Park proposed a model that could predict the stress-strain relation of rectangular hoop-confined concrete until 80% drop in the maximum stress in 1971. The ascending portion of the relation curve is identical for unconfined and confined concrete, which means that the increase in concrete strength is neglected and the confining effect is only expected to increase concrete ductility. In this model, the descending slope of the compressive stress after peak is determined by the volumetric ratio of lateral steel to concrete core, the ratio of hoop spacing to width of confined core, and the strength of plain unconfined concrete [1.44]. At the same year, Sargin investigated effects of rectilinear lateral reinforcement upon the strength and deformation properties of concrete and proposed a continuous function to depict the stress-strain relationship, which considered not only the variables considered in the Kent and Park model [1.44], the effects of confining steel strength and strain gradient on confinement were also taken into consideration [1.45]. In 1982, Park et al modified the Kent and Park model to take into account the enhancement of both concrete strength and ductility based on four nearly full-size concrete columns reinforced by rectilinear hoops [1.46].

Confining effects of reinforced concrete related to lateral steel reinforcement according to investigations so far. Vallenias et al. have found that longitudinal reinforcement also had some influence on confinement effectiveness based on the tests of fourteen prismatic specimens confined by rectangular hoops. The addition of longitudinal reinforcement could improve confinement effect, about 7% on average higher increase in compressive strength of concrete compared with that of specimens without longitudinal steel [1.47]. A few years later, Sheikh and Uzumeri explained effects of longitudinal rebars on confinement using the concept of effectively confined concrete areas within the nominal concrete core. They found that a portion of the core concrete also became less effective, which should be subtracted from the area of the effectively confined concrete when the cover concrete spalled off in the case of rectilinear reinforcement. The longitudinal rebar fully supported by hoop could restrain concrete effectively in its vicinity, which means that with the increase in the number of supported longitudinal rebars, the area of effectively confined concrete is also increased. Effects of distribution of longitudinal rebars were finally taken into account through center-to-center distance between longitudinal rebars in Sheikh and Uzumeri model [1.48, 1.49].

Besides for the rectilinearly hoop-confined concrete, complete stress-strain relations containing ascending and descending portions for the concrete confined by circular spirals or hoops have also been proposed by researchers [1.50-1.53]. Iyengar introduced a factor called the “confinement index” to define the confinement quantitatively. Ultimate strengths and strains increase with confinement and linearly with the confinement index [1.50]. Desayi proposed a stress-strain model of concrete confined in circular spiral. That model was determined by the plain concrete strength, the yield strength and volumetric ratio of spirals to concrete core [1.51]. Ahmad and Shah also investigated the stress-strain curve of concrete confined by spiral and added parameters of the ratio of spiral spacing to the width of concrete core into their model. Besides, Ahmad and Shah found that the use of lightweight aggregate decreased the effectiveness of spiral confinement [1.52]. It should be noted that although Ahmad and Shah’s model could give an accurate prediction of stress-strain relation of confined concrete, the calculating procedure is complex and needs a computerized method to realize the iterative algorithm. In 1985, Ahmad and Shah modified their model by simplifying calculating procedure and taking effects of the strain rate into consideration. According to the experimental results, there was an increase

in the secant modulus of elasticity, the peak stress, and the peak strain for hoop confined concrete under high strain rates [1.53].

Mander developed a unified approach applicable to all configurations of circular- and rectangular-shaped transverse reinforcement including spiral or circular hoops, or rectangular hoops with or without supplementary cross ties in 1988. Just like Sheikh and Uzumeri, Mander also considered the arching effect of steel rebars on confinement of concrete by using a confinement effectiveness coefficient. Mander's model indicates that effecting variables of circular spiral or hoops confinement were the plain concrete strength, the volumetric ratio of lateral steel to concrete core, the ratio of lateral reinforcement spacing to width of concrete core, while variables of rectangular hoop confined concrete further include configuration of longitudinal rebars, of which effects were reflected by the clear distance between adjacent supported longitudinal rebars. Because Mander's model could also be used to evaluate cyclic loading response and dynamic response, it has been widely adopted in numerical analysis [1.54, 1.55].

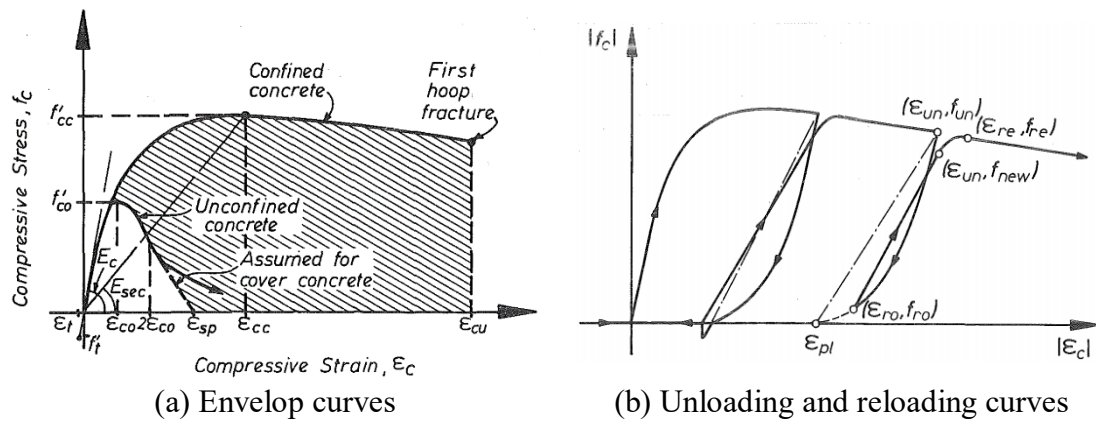


Fig. 1-8 Stress-strain curves for confined concrete proposed by Mander [1.54]

Besides the amount of lateral reinforcement (volumetric ratio of lateral steel to concrete core), the strength of steel and plain concrete, the hoop spacing or the ratio of hoop spacing to width of concrete core, the distribution of longitudinal rebars will affect stress-strain relations of confined concrete. In 1994, Sakino and Sun verified that the diameter of lateral steel bar was related to out of plane stiffness and could influence the confinement effectiveness by rectilinear hoops or steel tubes [1.56]. They introduced a factor, the ratio of lateral steel diameter to distance between adjacent effectively supported

longitudinal bars, to consider effect of the diameter or thickness of rectilinear bars or steel plates on confinement. The model proposed by Sakino and Sun has found a wide applicability to not only rectilinear hoop confined concrete but also square tube confined concrete with few simple equations [1.56]. Sakino and Sun further investigated effects of the strain gradient on the stress-strain characteristics of concrete confined by circular hoops or steel tubes to extend the range of application of the model to concrete members confined by transverse reinforcement with any shape and form under combined axial and flexural loading [1.57-1.63].

1.4 Problems and research objectives

As described above, in order for the concrete walls reinforced by CFRP bars to develop their self-centering capability, well confined concrete is indispensable in the boundary elements of the wall panels. However, studies on CFRP reinforcing walls are scarce and primarily focus on experimental investigation of walls under relatively low axial load level. To promote application of the concrete walls reinforced with CFRP bars into practice, several problems needed to be solved as listed below:

- 1) Seismic behavior of concrete walls reinforced by CFRP bars under high axial load: Concrete walls have much larger lateral stiffness than columns and are usually used in high-rise buildings, which inevitably cause high axial loads in the walls. As above mentioned, the previous studies conducted by Zhao et al. concentrated on experimental study of the walls under low axial load with axial load ratio of 0.1, and seismic behavior of CFRP reinforcing walls under high axial load is still unknown. Therefore, it is necessary to investigate seismic behavior of walls under high axial load.
- 2) Seismic behavior of CFRP reinforcing walls with improved confinement: According to the experimental results by Zhao et al., the maximum drift of CFRP reinforcing walls confined by lateral rebars with nominal diameter of 10mm at a spacing of 100mm is 1.4% that is not sufficient to guarantee the needed deformability for high-rise buildings under great earthquakes. It has been well known that decreasing the spacing of transverse hoops and/or improving their configuration can enhance the strength and ductility of confined concrete. Therefore, it can be theoretically speculated that using close hoops and reasonable configuration will strengthen deformability of concrete walls, which needed to be verified and quantitatively analyzed based on experimental results.
- 3) Evaluation method of seismic performance of CFRP bars reinforcing walls: Previous studies on concrete walls reinforced by CFRP bars are mainly experimental investigations and there has not been a reliable and accurate assessing method to predict hysteretic characteristics, neither calculating equations of lateral resistance

and deformation of CFRP reinforcing walls has been proposed so far.

Objectives of this doctoral dissertation are to address above-mentioned three problems, and described as follows:

- 1) to conduct experimental investigations on seismic behavior of CFRP reinforcing walls subjected to the high axial load with axial load ratio of up to 0.33.
- 2) to experimentally verify enhancing effects of improved confinement on seismic behavior of concrete walls reinforced by CFRP bars and obtain fundamental information that is necessary as developing evaluation method for seismic behavior of the walls.
- 3) to present methods of evaluating the ultimate lateral resistance capacity as well as of predicting overall seismic behavior of concrete walls reinforced by CFRP bars.

1.5 Format of the dissertation

This dissertation includes seven chapters. The first chapter introduces backgrounds of this study and reviews relevant studies conducted by other researchers to find problems and derive researching objectives. Besides, the contents of each chapter are also introduced briefly in this chapter.

Chapter two describes the experimental investigation on concrete walls reinforced by CFRP bars with boundary elements in each wall panel confined by rectilinear hoops. Three full-scale concrete walls having the same reinforcement configuration were fabricated and tested under reversed cyclic lateral loads and constant axial load with axial load ratios of 0.17, 0.26, 0.33, respectively. The boundary elements were confined by composite rectilinear hoops with a nominal diameter of 6mm and having close spacing of 50mm aimed at enhancing confinement effect and improve overall seismic behavior of the walls. Besides the boundary elements, the potential hinge region of the wall panels may also be subjected to large deformation and damage under reversed cyclic lateral loading. To mitigate potential damage of the wall panels, horizontal distributed reinforcement in the wall panels is also placed with close spacing of 50mm within the potential plastic region. Eight CFRP bars were placed as longitudinal reinforcement at each boundary element. Objectives of this chapter are to investigate effects of axial load levels on seismic behavior of CFRP reinforcing walls and verify if the improved confining methods can ensure sufficient seismic performance, especially deformability to concrete walls.

Chapter three presents the experimental information on seismic performance of walls reinforced with CFRP bars, having the same experimental variable, the axial load ratio, as that of test walls described in chapter two. Three full-scale concrete walls having the same geometry and reinforcement details except the configuration of transverse hoops in the boundary elements of wall panels as compared with specimens described in the previous chapter. The boundary elements of these specimens were confined with circular hoops. Objective of the experiments conducted here is to investigate effect of confinement by circular hoops on the seismic behavior of CFRP reinforcing walls. As reviewed before, confining effects of circular hoops are more efficient than that of rectilinear hoops, so

using circular hoops may achieve the same confining level as the composite rectilinear hoops. Experimental results will be used to verify the effectiveness of circular hoops.

Chapter four presents an integrated analytical method to assess hysteretic behavior of concrete walls reinforced with CFRP bars. The analytical method proposed by Sun et al. for evaluating seismic behavior of concrete columns reinforced by SBPDN rebars [1.32,1.22] is modified to trace cyclic behavior of concrete walls reinforced by CFRP bars by utilizing bond-slip relations for CFRP bars. The analytical results calculated by the proposed methods are compared with the experimental results described in chapter two and chapter three to verify reliability and accuracy of the proposed numerical analytical method.

Chapter five addresses evaluation of ultimate flexural and shear capacities of concrete walls reinforced with CFRP bars. Representative design equations recommended in current codes and/or standards are utilized to predict ultimate capacities of concrete walls with CFRP bars to investigate their applicability. Objective of this chapter is to find the most appropriate design formulae enable for engineers to conduct simple and prompt ultimate capacity design for the CFRP reinforcing walls through comparison between the calculated and experimental results.

Chapter six proposes a V-R (lateral force V versus drift R) envelop model for concrete walls reinforced by CFRP bars. The V-R envelop curve is indispensable in the performance- or drift-based seismic design of concrete components. Besides, the V-R envelop model can provide information about the deformation corresponding to respective ultimate capacity of concrete walls, hence helping engineers to reasonably assess degree of potential damage under various levels of earthquakes. The accuracy of the proposed V-R envelop model will be verified by comparing the results obtained by experiments and those calculated by numerical analysis described in chapter four.

As the last chapter of this thesis, chapter seven summarizes observations and findings obtained in chapter two through chapter six. Future works concerning with the concrete walls reinforced by CFRP bars are also presented at the end of this chapter.

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CHAPTER TWO

2. Seismic behavior of concrete walls reinforced by CFRP bars with boundary elements confined by rectilinear hoops

2.1 Introduction

Concrete walls have been widely used in building structures, especially in high-rise buildings, as lateral seismic resistant components because of their large lateral stiffness. Concrete walls located in seismic-prone regions are subjected to not only lateral earthquake-induced loading but also vertical loading from the self-weight of structures when hit by earthquakes, and the wall panel at the lowest story of high-rise buildings usually suffers high axial load. Therefore, seismic behavior of concrete walls under high axial load level is of fundamental significance for the whole building safety.

Boundary elements in the wall panel of concrete walls sustain the largest tension or compression stress under composite loading effects caused by vertical and lateral forces during earthquakes. Sufficient reinforcement is generally placed in the boundary elements to obtain sufficient compressive and tensile strength, resulting in high flexural strength of concrete walls. For resilient concrete walls, un-bonded post-tension steel bars and WBUS rebars are adopted as longitudinal reinforcement in the boundary elements. Transverse reinforcement in the boundary elements also plays a very important role in ensuring sound ductility to the compressive concrete by providing lateral confining stress to the concrete.

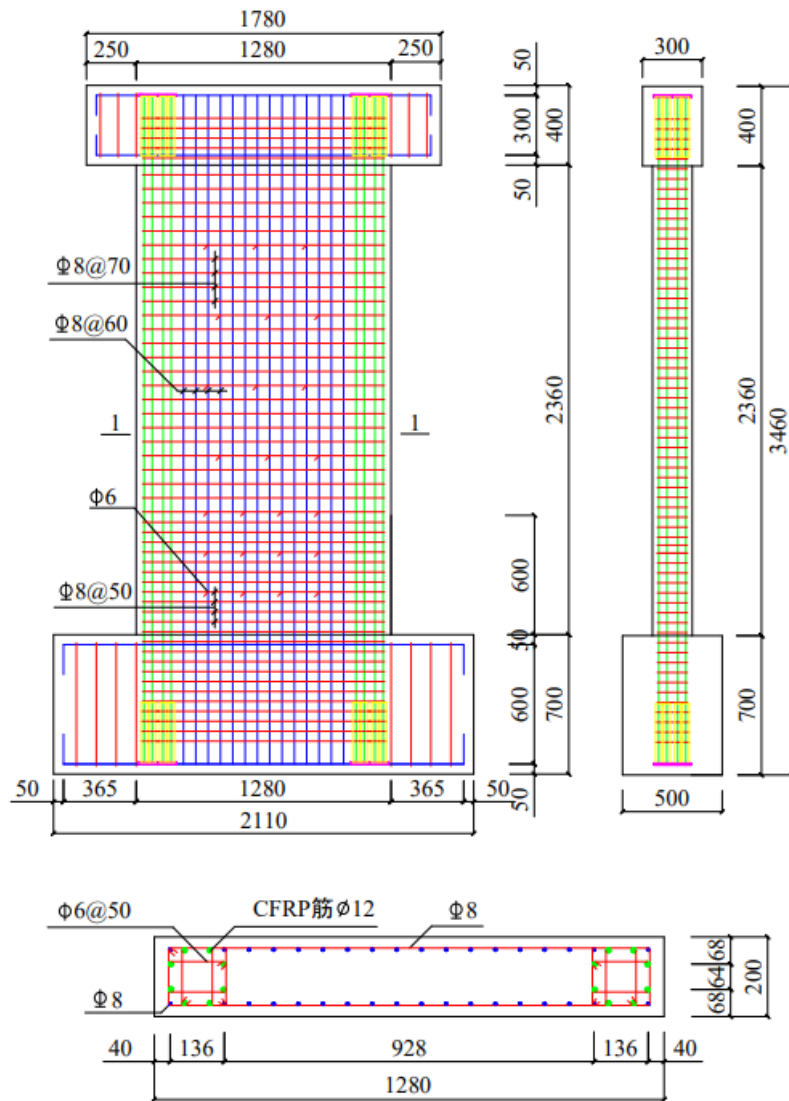
Objectives of this chapter are: 1) to obtain basic information on the cyclic behavior of concrete walls reinforced by CFRP bars under relatively high axial load with axial load ratio of up to 0.33, 2) to investigate the effects of axial load level on seismic behavior of

CFRP reinforcing concrete walls, and 3) to verify the effectiveness of composite rectilinear hoops with close spacing.

2.2 Experimental program

2.2.1 Details of test walls

For the purpose of investigating the seismic behavior concrete walls reinforced by CFRP bars with boundary elements confined by rectilinear hoops under different axial load ratios, three identical full-scale specimens were designed, fabricated and tested under the constant axial compression, which consists of three different axial load ratios varied from 0.17 to 0.33, and the reversed cyclic lateral load.



Section 1-1

Fig. 2-1 Dimensions and reinforcement details of test walls

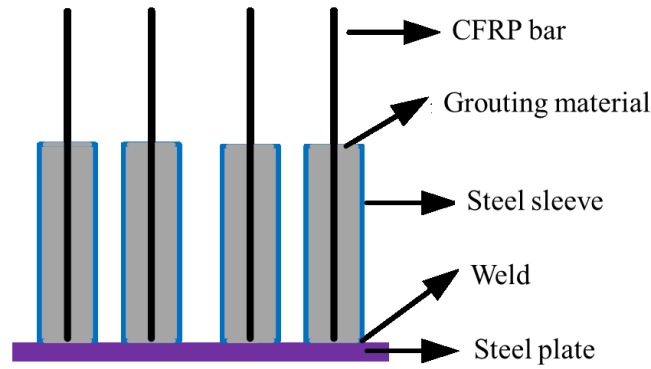


Fig. 2-2 Anchorage for CFRP bar

Table 2-1 Outlines of test specimens

Notation	L/l_w	n	Boundary element		Web	
			Longitudinal bars	Hoops	Vertical bars	Horizontal bars
CFRP RW1	2	0.17	8-CFRP bars 4-HRB335 bars	HPB300 @50 $\rho_h=3.9\%$	HRB335 @62 $\rho_v=0.8\%$	Region above the height of 600mm: HRB335@70 $\rho_w=0.7\%$
CFRP RW2		0.26				Region below the height of 600mm: HRB335@50 $\rho_w=1.0\%$
CFRP RW3		0.33				

Notes: L/l_w , shear span ratio; n , axial load ratio, $n=P/(Af'_c)$; P , axial load; A , sectional area of wall; f'_c , axial compressive strength of concrete; ρ_h , volumetric ratio of hoops to the confined core measured center to center of perimeter hoop of boundary element; ρ_v , steel ratio of vertical web reinforcement; ρ_w , steel ratio of horizontal web reinforcement.

All specimens had the same identical geometry and reinforcement details as shown in Fig.2-1. The wall panel was 2360mm high by 1280mm long and 200mm thick. The upper and lower loading beams were fabricated to apply loading force. The lateral loading point was at the height of 2560mm from the bottom of wall panel resulting in a shear span ratio of 2.0.

Eight CFRP bars with the diameter of 12mm were placed in each boundary element and plain steel rebars (HPB300) with the diameter of 6mm were used as composite rectilinear hoops at a spacing of 50mm to confine boundary elements. Web reinforcement of a wall

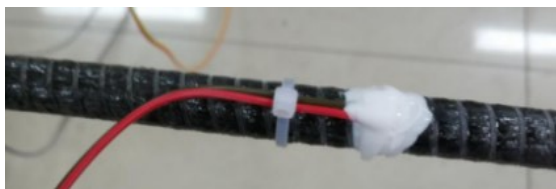
consisted of two layers deformed steel bars (HRB335) with the diameter of 8mm. Horizontal web rebars were placed at the spacing of 50mm within the height of 600mm (the potential plastic hinge region) resulting in a steel amount ratio of 1.0%, while the spacing of horizontal web rebars became 70mm at the upper region to give a steel amount about 0.7%. The vertical web reinforcement was placed uniformly at the spacing of 62mm that led to a steel amount ratio of 0.8%.

The anchoring equipment of CFRP bar is shown in Fig.2-2. Both ends of each CFRP bar were anchored to a 10mm-thick steel plate to which the steel sleeve with the length of 330mm housing CFRP bar with high-strength and non-contract grouting material was welded to avoid slippage and ensure the full performance of CFRP bars.

As shown in the Table 2-1, The experimental variable was the axial load level represented as axial load ratios of 0.17, 0.26 and 0.33. Specimen numbers of CFRPRW1, CFRPRW2 and CFRPRW3 represent three test walls under axial load ratios of 0.17, 0.26 and 0.33 and the corresponding axial loads were 1580kN, 2431kN, 3160kN, respectively.

2.2.2 Materials properties

CFRP bars have the different surface shape from deformed steel bars as shown in Fig. 2-3. Unlike crescent ribs on the surface of deformed steel bars, spiraled grooves can be seen on the surface of CFRP bars, which results in different bound-slip characteristics of bars in concrete. Table 2-2 summarizes mechanical properties of used CFRP bars and steel bars. The ultra-high tensile strength and completely linear elasticity of CFRP bars are also different from ordinary steel bars that have yield stage as shown in Fig. 2-4.



CFRP bar



Deformed steel bar

Fig. 2-3 Surface shapes of bars

Ready-mixed concrete was used to fabricate the concrete walls and its measured cubic

(150mm) compressive strength, f'_{cu} , was 57.2MPa. The axial compressive strength, f'_c , that is usually used to assess behavior of concrete structures in Chinese design code, such as the axial load ratio is defined based on the axial compressive strength, f'_c , that definition was adopted in this dissertation. The value of f'_c can be obtained from f'_{cu} using the Eq. (2.1) [2.1]. The calculating f'_c equals 36.8MPa.

Table 2-2 Mechanical properties of reinforcement

Notation	d_b (mm)	A_s (mm ²)	f_y (MPa)	f_u (MPa)	ε_y ($\mu\varepsilon$)	δ (%)	E_s (GPa)
HPB300	6	28.3	414.8	529.4	1860	19.4	223
HRB335	8	50.3	361.3	502.6	1599	19.2	226
CFRP bar	12	113.1	-	2310.3	-	-	143

Notes: d_b , diameter; A_s , cross-sectional area; f_y , yield stress; f_u , ultimate strength; ε_y , yield strain; δ , breaking elongation; E_s , Young's modulus.

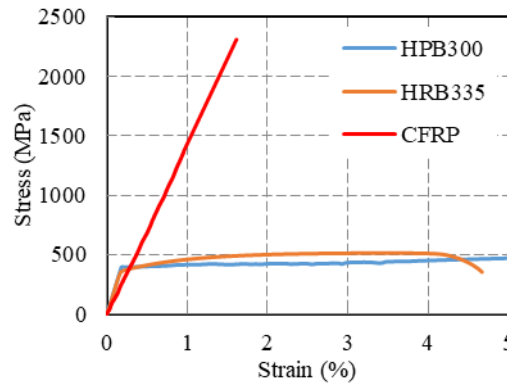


Fig. 2-4 Stress-strain curves of CFRP and steel reinforcement

$$f'_c = 0.88\alpha_{c1}\alpha_{c2}f'_{cu} \quad (2.1)$$

Where:

$$\alpha_{c1} = 0.76 \text{ for } f'_{cu} \geq 50 \text{ MPa}$$

$$\alpha_{c1} = 0.82 \text{ for } f'_{cu} = 80 \text{ MPa}$$

$$\alpha_{c2} \text{ varies linearly between 0.76 and 0.82 for } 50 \text{ MPa} < f'_{cu} < 80 \text{ MPa}$$

$$\alpha_{c2} = 0.76 \text{ for } f'_{cu} \geq 50 \text{ MPa}$$

$$\alpha_{c2} = 0.82 \text{ for } f'_{cu} = 80 \text{ MPa}$$

$$\alpha_{c2} \text{ varies linearly between 0.76 and 0.82 for } 50 \text{ MPa} < f'_{cu} < 80 \text{ MPa}$$

2.2.3 Loading apparatus

Walls were tested under reversed cyclic lateral loading and constant axial load applied by the loading apparatus shown in Fig. 2-5. The reversed cyclic lateral load was applied to the upper loading beam of specimen by using an MTS servo-controlled hydraulic actor with the maximum capacity of 2500kN. There were two vertical hydraulic jacks each with the capacity of 2000kN to provide the designed axial load level to test specimens. A sliding mechanism between vertical hydraulic jacks and reaction steel beam is also necessary to let jacks to move with the loading beam of specimen under the cyclic lateral load. Besides, Axial loads could be adjusted automatically in real time by hydraulic oil pump during the whole test to keep the constant axial load level for test walls.



Fig. 2-5 Loading apparatus

The reversed cyclic lateral load was controlled by the drift ratio that was calculated

through dividing the lateral displacement of MTS servo-controlled actor by the shear span. The loading program is illustrated in Fig. 2-6. Two complete cycles were applied at each drift level with a increment of 0.2% up to the drift of 2.0%, after which the reversed cycle became one cycle at specified drifts and the increment of drift between adjacent levels reached 0.5%.

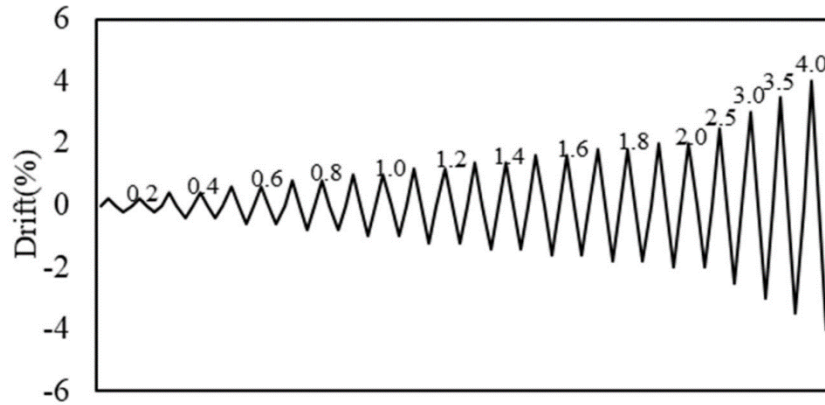


Fig. 2-6 Loading program

2.2.4 Instrumentation

As shown in Fig. 2-7, the actual lateral and axial displacement of each test wall were measured through displacement transducers (DT). One DT was set at the center of upper loading beam to obtained the actual lateral displacement of specimen that is usually different form the displacement of actor because of inevitable interspaces and other affecting factors. Three pairs of DT were used to measure the axial deformation within different gauge lengths along the wall height. And Four pairs of lateral DT were installed to measure the distribution of lateral deformation along height at specified drift levels.

To measure the strains of CFRP bars and hoops in boundary elements, strain gauges were embedded onto the surface of rebars as shown in Fig. 2-8. The strains of horizontal and vertical web steel bars were also measured by gauges, of which locations are shown in Fig. 2-9.

Chapter 2

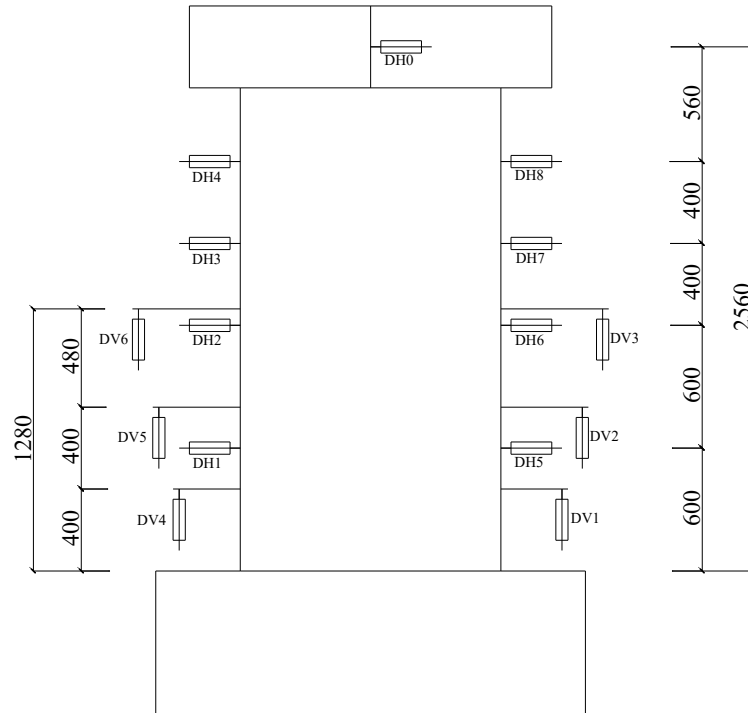


Fig. 2-7 Locations of displacement transducers

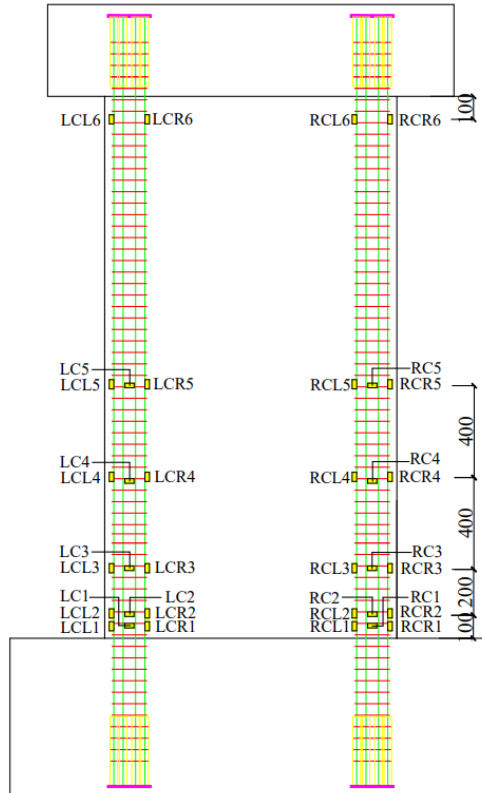


Fig. 2-8 Locations of strain gauges of CFRP bars and hoops in boundary elements

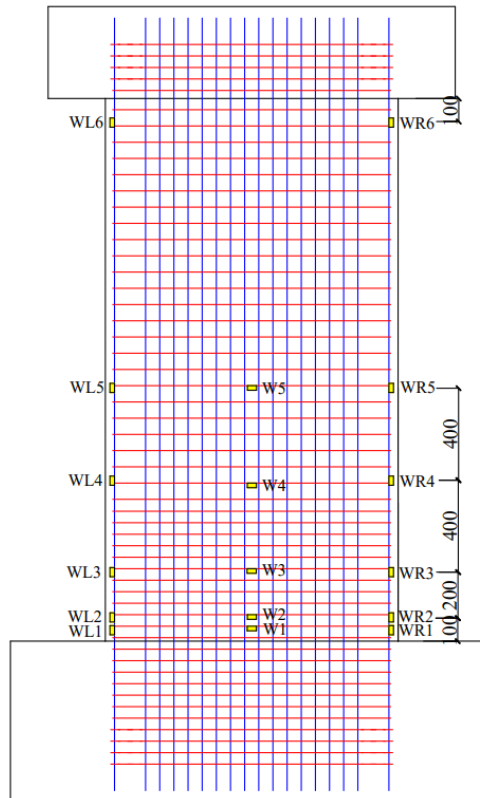


Fig. 2-9 Locations of strain gauges of web reinforcement

2.3 Test results and observation

2.3.1 Observed behavior

Fig. 2-10 through Fig. 2-12 show the crack patterns of three specimens at several specific drifts. It should be mentioned that the lateral loading program were controlled according to the actor displacement, while the actual displacement of specimen was always not equal with that displacement. So the actual drift calculated using specimen displacement are also given in brackets behind the drift derived from actor displacement. The last drift shown in each figure means the final loading cycle of each specimen, respectively. The red and blue lines in figures represent cracks observed in push and pull directions, respectively. The square grids represented by dotted lines with the side of 100mm are adopted as a reference to clearly indicate the propagation of crack. In addition, the spalling of concrete shell was shown by the solid block with black color.

For specimen CFRPRW1, the first flexural crack was observed at the drift of 0.06% at the section 300mm away from the lower stub in push direction, after which two flexural cracks in pull direction commenced at the section 250mm and 430mm away from the lower stub when the drift reached 0.09%. The flexure-shear crack was first detected at the section 650mm away from the base beam of specimen at the drift of 0.18%. Cracks propagated along with increasing drift. When the drift increased to 1.29%, cover concrete started to flake. At the end of test, cracks sufficiently propagated along whole height of the wall and cover concrete spalling could be observed to concentrate at toes of the wall panel as shown in Fig. 2-10.

CFRPRW2 was tested under a larger axial load than CFRPRW1. Two flexural cracks on CFRPRW2 were first observed in push direction at the section 350mm and 810mm away from the lower stub, respectively as increasing the drift up to 0.15%, while the lateral force reversed to pull direction, three flexural cracks were confirmed at the section 150mm, 200mm, 500mm away from base with 0.08% drift. The first flexure-shear crack formed at the section 500mm away from the lower stub as the drift reached 0.27%. The cover concrete spalling off started from 0.89% drift that was earlier than CFRPRW1 result from the larger axial load. The effect of axial load on crack propagation can be seen in

Fig. 2-11 that the crack pattern of CFRPRW2 exhibits the lower height of 1750mm than that of specimen CFRPRW1 at the end of test.

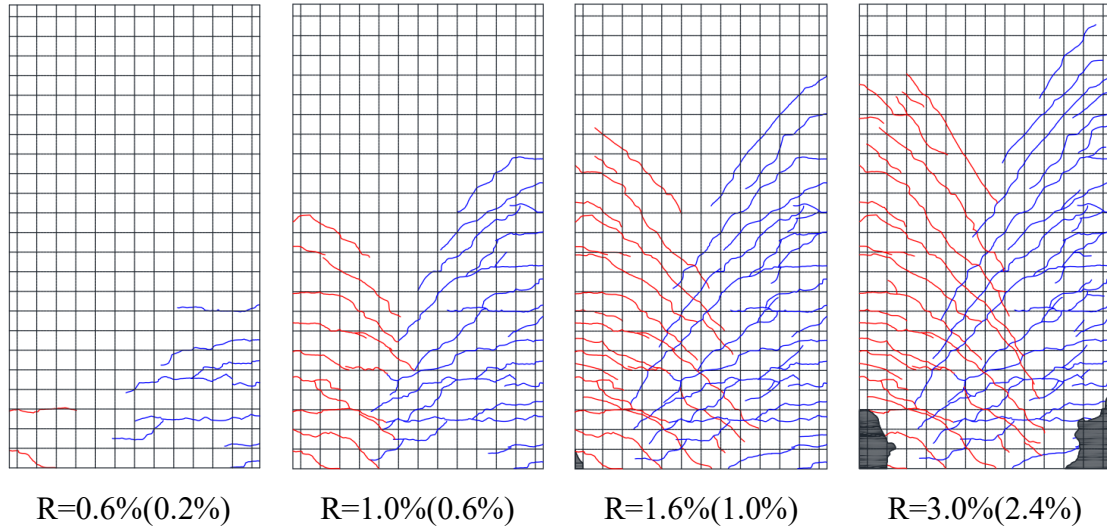


Fig. 2-10 Crack patterns of specimen CFRPRW1

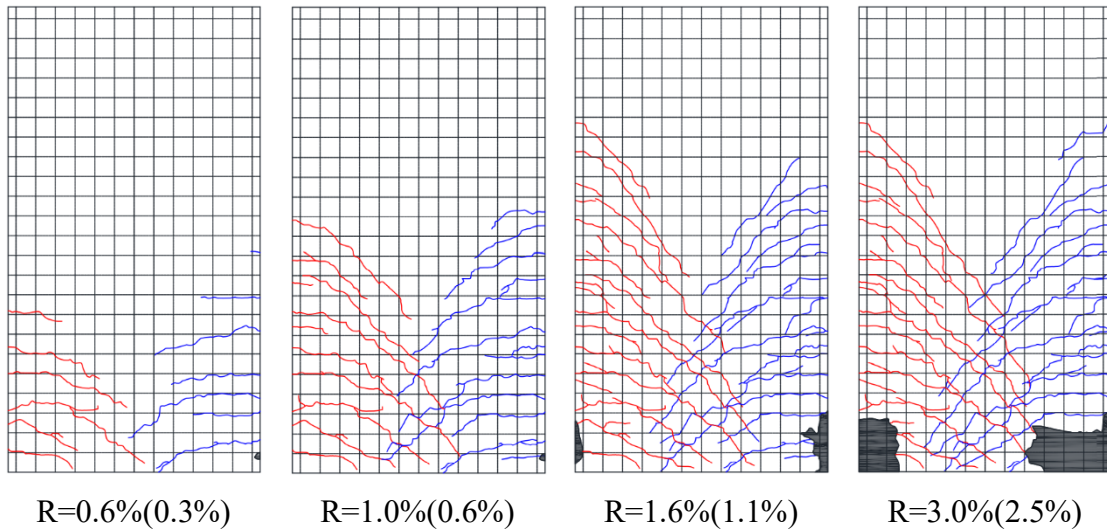


Fig. 2-11 Crack patterns of specimen CFRPRW2

For specimen CFRPRW3 that tested under the largest axial load ratio of 0.33, the first bending cracks in push and pull directions were noticed at drifts of 0.18% and 0.16%, respectively. These flexural cracks distributed at the heights varied from 150mm to 480mm. While the crack began propagating diagonally to flexure-shear crack at 0.33% drift. The cover concrete spalling started at the drift of 0.44% that was the earliest among three specimens. Besides, the height of final crack pattern of CFRPRW3 was the lowest

with the value of 1600mm.

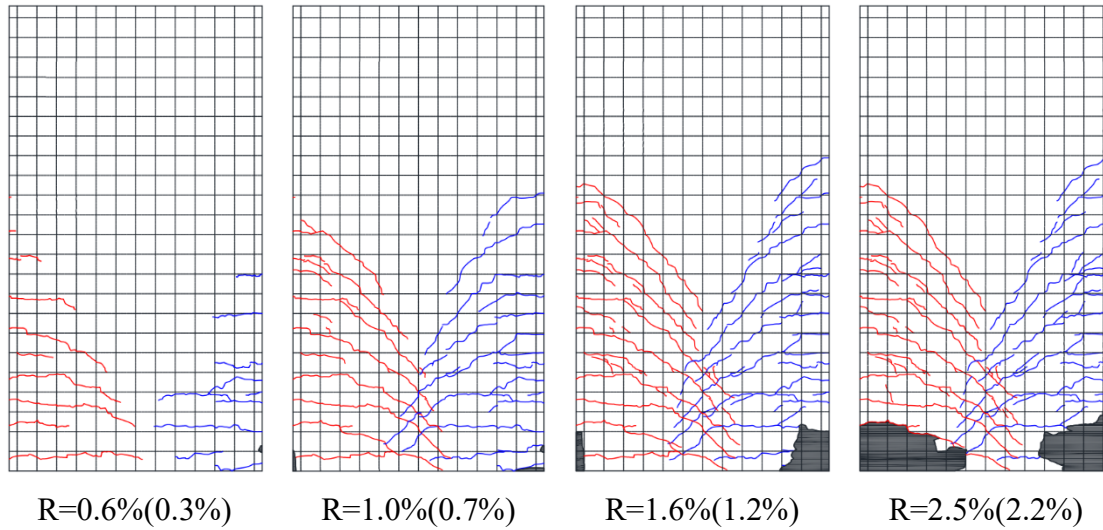


Fig. 2-12 Crack patterns of specimen CFRPRW3

As described above, the axial load ratio has effects on cover concrete spalling and crack propagation, which can be concluded that cover concrete spalling started at relatively little drift as increasing the drift ratio, meanwhile the crack propagation was restrained.

2.3.2 Lateral force-drift responses

The lateral force-drift (V-R) responses of all specimens are shown in Fig.2-13. The superimposed solid red circles and dotted black lines represent the measured maximum load and the P- Δ effect by the axial load on lateral resistance, respectively. Experimental values of the maximum lateral force and the corresponding drift are summarized in Table 2-3.

As can be seen from Fig. 2-13, All specimens had the so-called origin-pointing hysteretic loops, which means that the V-R curve approached toward origin at unloading stage indicating small residual drift. Specimens CFRPRW1 and CFRPRW2 exhibited stable lateral resistance even at the large drift beyond 2.0%. Specimen CFRPRW3 also showed the well lateral resistance tell 1.2% drift, after which the degradation of lateral force was observed due to the crushing of concrete under high load ratio of 0.33 as well as the compression failure of CFRP bars.

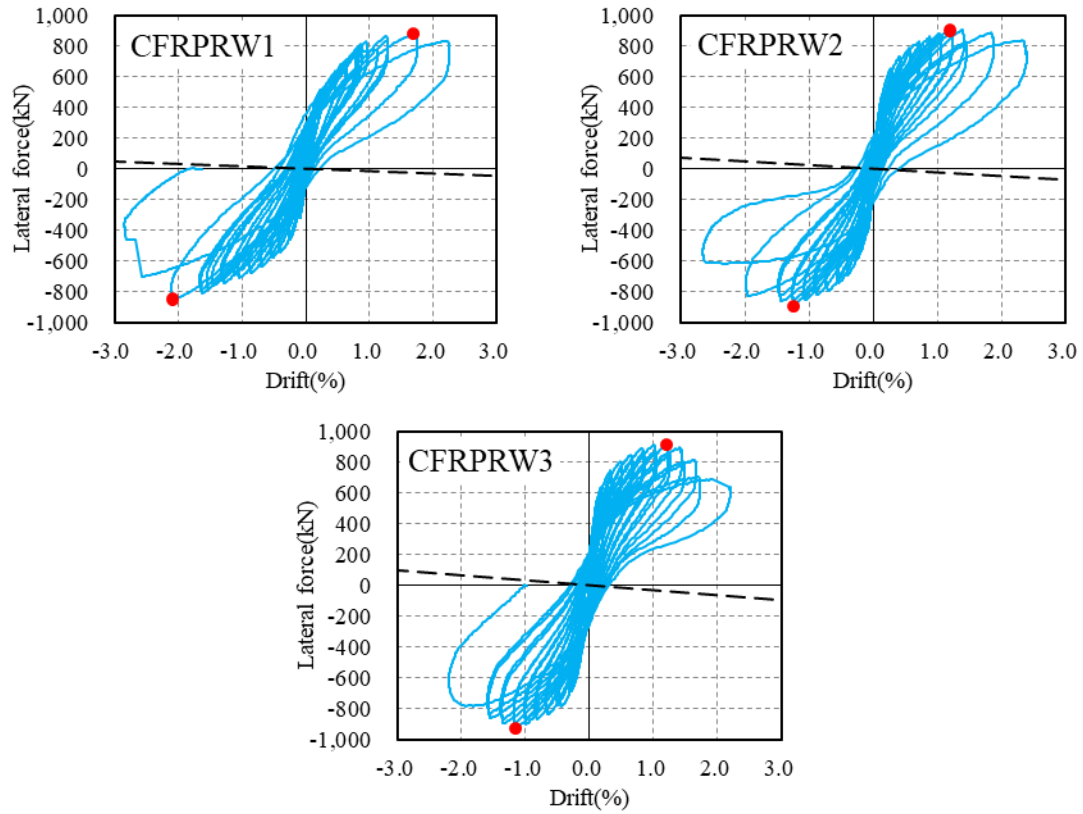


Fig. 2-13 Measured lateral force-drift (V-R) relationships of specimens

Table 2-3 Primary results of specimens

Notation	n	V_{max} (kN)	R_{max} (%)	V_{min} (kN)	R_{min} (%)	V_{ave} (kN)	R_{ave} (%)
CFRPRW1	0.17	888.77	1.74	-855.49	-2.10	872.13	1.92
CFRPRW2	0.26	914.72	1.22	-880.24	-1.23	897.48	1.23
CFRPRW3	0.33	920.52	1.21	-923.60	-1.15	922.06	1.18

Notes: n , axial load ratio; V_{max} , peak lateral force in push direction; R_{max} , corresponding drift to V_{max} ; V_{min} , peak lateral force in pull direction; R_{min} , corresponding drift to V_{min} ; V_{ave} , average of V_{max} and V_{min} ; R_{ave} , average of R_{max} and R_{min} .

The envelop curves of V-R relationships are shown in Fig. 2-14 to better understand the effect of axial load ratio. The envelop curves are obtained by connecting the peak point of the first cycle of each drift level using straight lines. It should be mentioned that data that lateral load declined to below 85% maximum lateral load was removed according to the Specification for Seismic Test of Buildings of China (JGJ/T 101-2015) [2.2].

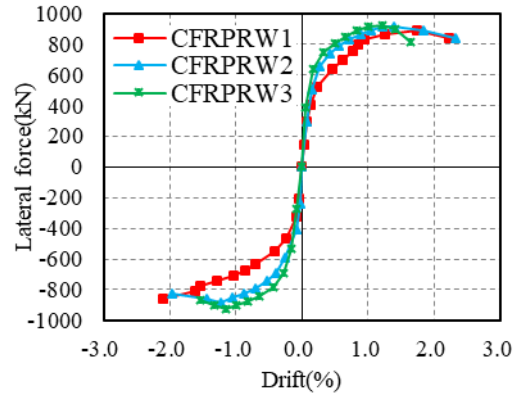


Fig. 2-14 Comparison of envelop curves

As can be seen from Table 2-3 and Fig. 2-14, The axial load has effects on V-R relationships that the lateral force of specimen under high axial load ratio was larger than that under relatively lower axial load at same specified drifts before the respective peak value of lateral force. After that, the lateral force of CFRPRW3 degraded more sharply than other specimens result from the concrete spalling and compression failure of CFRP bars under the high axial load ratio. Besides, the peak lateral force increased as increasing the axial load ratio, while the corresponding drift and ultimate drift showed the opposite tendency. CFRPRW2 and CFRPRW3 experienced larger maximum lateral loads with increments of 2.9% and 5.7%, respectively, compared with CFRPRW1. The ultimate drift was also affected by the axial load ratio that the high axial load level with the ratio of 0.33 decreased the ultimate drift of CFRPRW3 by 35.6% compared with CFRPRW1 and CFRPRW3.

2.3.3 Stiffness Degradation

The stiffness reflects capacity of resisting deformation. For concrete walls, the stiffness before cracks commencing is initial stiffness, which indicates the deformation-resisting capacity during the elastic stage. As mentioned in the section 2.3.1, there was no cracks observed on all specimens at the first loading cycle. So the experimental initial stiffness can be obtained using the measured peak lateral load and corresponding displacement at the end of first loading cycle. The calculating equation suggested by JGJ/T 101-2015 [2.2] is shown as below:

$$K_i = \frac{|+V_i| + |-V_i|}{|+\Delta_i| + |-\Delta_i|} \quad (2.2)$$

Where, K_i is the secant stiffness of i -th loading cycle, $+V_i$ and $-V_i$ are the peak lateral force of i -th loading cycle in push and pull directions, respectively. $+\Delta_i$ and $-\Delta_i$ are the corresponding lateral displacement to $+V_i$ and $-V_i$, respectively. Eq. (2.2) can be used to calculate not only the initial stiffness of concrete wall but also the secant stiffness of different drift levels after cracks commencing when the concrete walls turn from elasticity into plasticity.

Because the initial stiffness means the stiffness of concrete wall in elastic stage, the initial stiffness can also be obtained by Eq. (2.3) through Eq. (2.6) on basis of elastic theory.

$$K_e = V/\Delta \quad (2.3)$$

$$\Delta = \Delta_B + \Delta_S \quad (2.4)$$

$$\Delta_B = VL^3/(3E_c I_e) \quad (2.5)$$

$$\Delta_S = \kappa VL/(G_c A) \quad (2.6)$$

Where, K_e is the initial stiffness, V is the lateral force, Δ is the lateral displacement, Δ_B and Δ_S are the lateral displacement caused by the flexural and shear deformation, respectively, L is the shear span, E_c is the elastic modulus of concrete (N/mm²) that can be obtained by Eq. (2.7) from compressive strength according to reference [2.1], I_e is the second moment of area considering reinforcement, κ is the form factor (=1.2), G_c is the shear modulus of concrete (=0.4 E_c [2.1]), A is the sectional area of concrete wall.

$$E_c = \frac{10^5}{2.2 + \frac{34.7}{f'_{cu}}} \quad (2.7)$$

The experimental initial stiffness and calculated initial stiffness of each specimen are summarized in Table 2-4. The calculating value of initial stiffness obtained from Eq. (2.2) through Eq. (2.7) can give a reasonable predication of experimental initial stiffness within the error range of 5% ~ 11% for three specimens.

As consecutive lateral loading continued, the stiffness of concrete wall degraded after cracking because of the accumulation of damage. The secant stiffness at the first cycle of each drift level is calculated by Eq. (2.2) and shown in the form of ratio to experimental

initial stiffness (K_i/K_1) in Fig. 2-15 to indicate the stiffness degradation characteristic along with increasing drift. The stiffness of all specimens degraded radically before the drift of 0.5%, after which the stiffness degradation tend to slow down and declined to about 5% of initial stiffness. It can also be seen in Fig. 2-15 that the high axial load ratio decreased the degree of stiffness degradation.

Table 2-4 Comparison between experimental and calculating initial stiffness

Notation	Experiment	Calculation				K_1/K_e
	K_1 (kN/mm)	K_e (kN/mm)	Δ_B (mm)	Δ_S (mm)	Δ_S/Δ	
CFRPRW1	209.78	199.36	4.17	0.84	0.17	1.05
CFRPRW2	222.19					1.11
CFRPRW3	184.22					0.92

Notes: K_1 , experimental initial stiffness; K_e , calculating initial stiffness; Δ_B , lateral displacement caused by flexural deformation under the lateral loading of 1N; Δ_S , lateral displacement caused by shear deformation under the lateral loading of 1N; Δ , $\Delta_B + \Delta_S$.

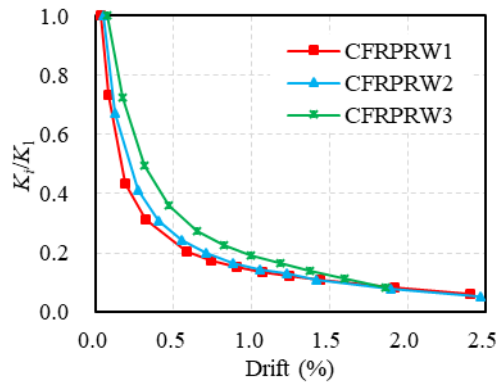


Fig. 2-15 Comparison of the stiffness degradation

2.3.4 Strains measured in steel bars and CFRP bars

The strains of CFRP bars measured at the section 100mm away from the wall base are shown in Fig. 2-16 to indicate strain and stress states of CFRP bars reinforcing in concrete walls under the quasi-static lateral load. The blue continuous lines represent the strains of CFRP bars, while the green dotted horizontal lines are utilized to show the ultimate tensile strain of the CFRP bar.

It can be observed that CFRP bars did not reached the ultimate tensile strain in spite of the axial load ratio. While CFRP bars may suffer compression failure under the high compression, which was more obvious in the case of high axial load ratio. The compression failure of CFRP bar in CFRPRW3 occurred at the drift of 1.3% as shown in Fig. 2-16. The compression failure of CFRP bars can account for the decline of lateral resistance of wall. Fig. 2-16 also shows that the strain-drift relations of CFRP bars exhibit the same origin-oriented characteristic like V-R curves of walls, which means that the ultra-high tensile strength with completely linear elasticity of CFRP bars provides resilience for concrete walls.

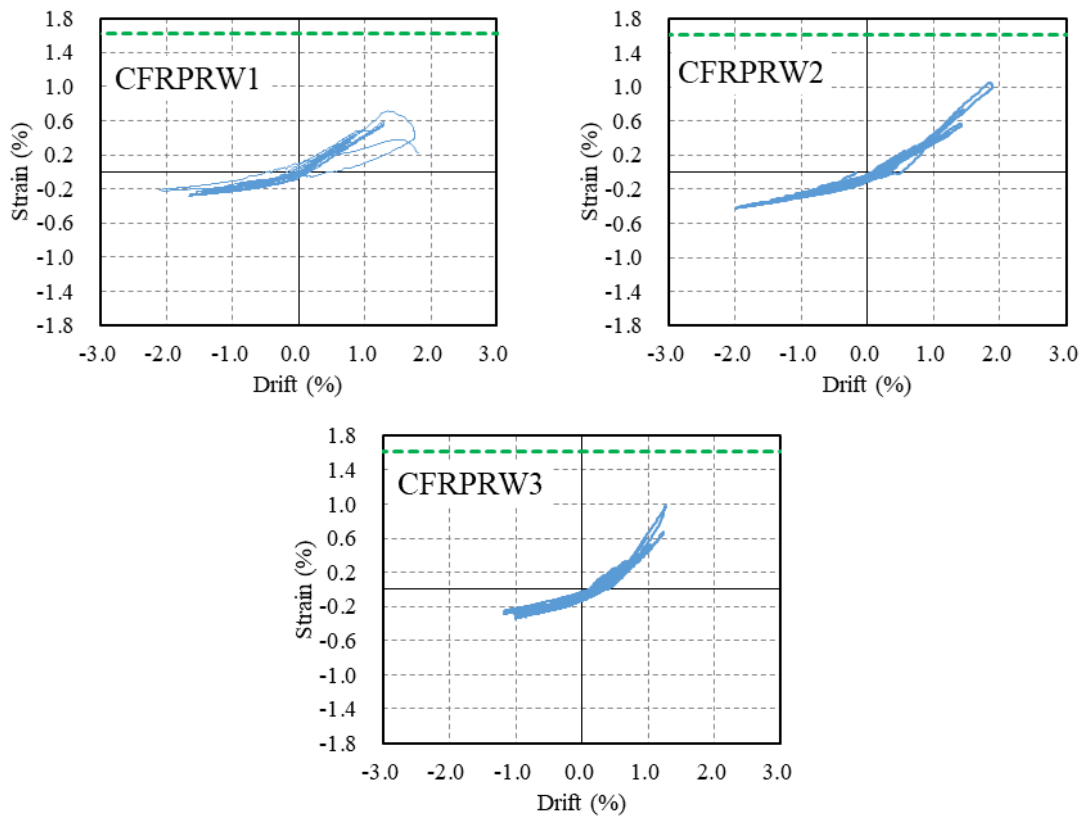


Fig. 2-16 Measured strains of CFRP bars

Fig. 2-17 shows the strain profiles of CFRP bars along wall height at several drifts, where height means distance from the wall base. The strains distribution of CFRP bar along the wall height indicate characteristic that the strain measured in the potential hinge region exhibited a nearly uniform distribution resulting from the slippage of CFRP bar in concrete.

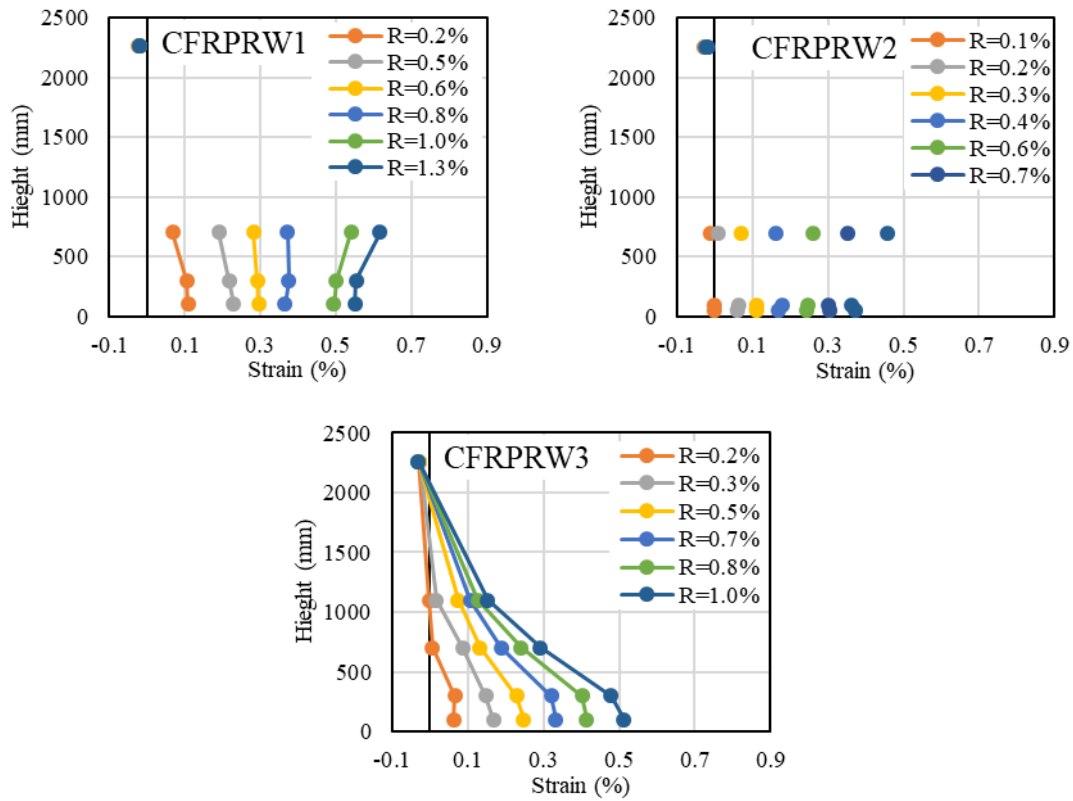


Fig. 2-17 Strains distribution of CFRP bar

Fig. 2-18 indicates the strains of longitudinal steel bars placed in boundary elements at the height of 100mm. The superposed green dotted lines represent the yield strain of steel bars. These ordinary deformed steel bars reached yield strain early before 0.5% drift, after which generated plastic deformation of steel bar gave a negative effect on the resilience of walls.

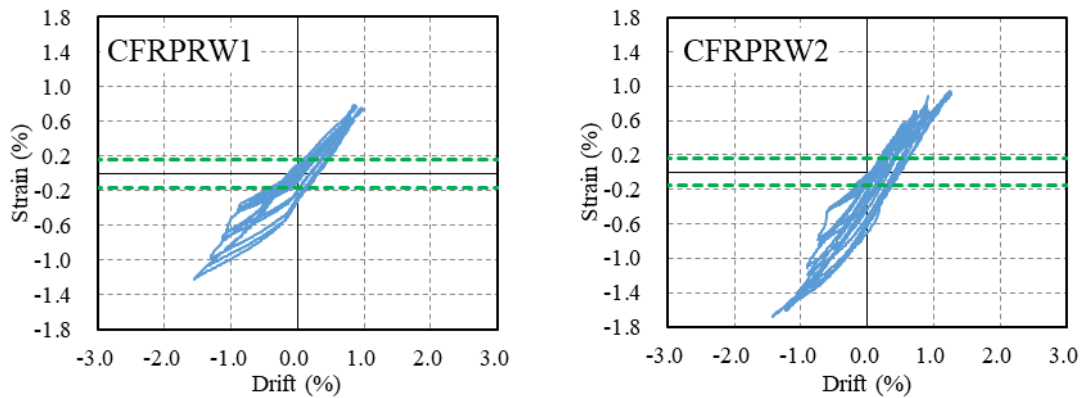


Fig. 2-18 Measured strains of longitudinal steel bars

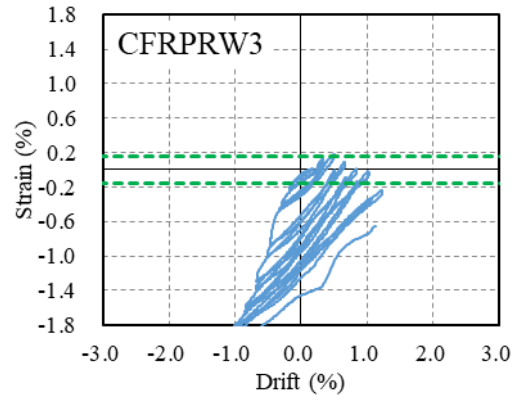


Fig. 2-18 Continued

Fig. 2-19 shows the strains of hoops in boundary elements at the height of 100mm using blue solid lines. The green dotted lines in this figure also represent the yield strain of steel hoops. Hoop strains of CFRPRW2 and CFRPRW3 reached yield state at the drifts of 1.5% and 1.2%, respectively because of the relatively high axial load ratio, while the hoop of CFRPRW1 tested under the lowest axial load ratio of 0.17 did not yield even increasing the drift up to 2.0%.

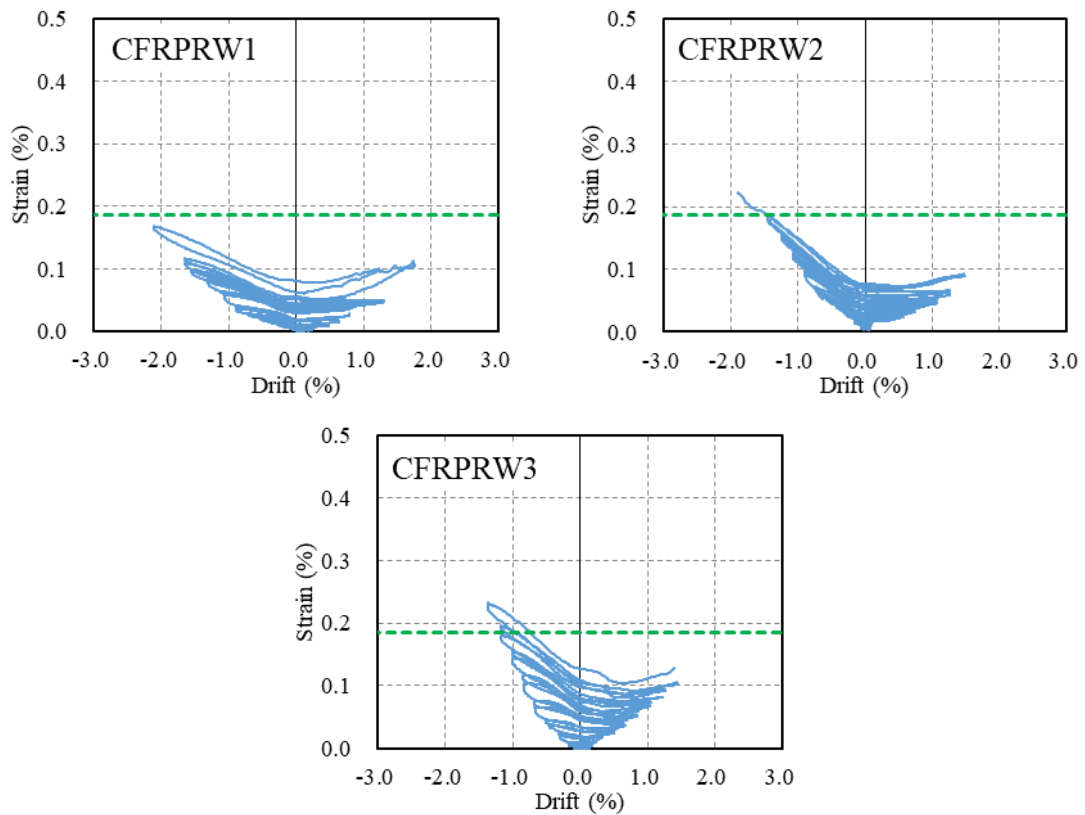


Fig. 2-19 Measured strains of hoops in boundary elements

The strains of horizontal web reinforcements measured at the height of 100mm are shown in Fig. 2-20, of which yield strain is represented by green dotted line. It can be seen that only the horizontal web steel bar of CFRPRW3 reached yield strain at the 1.0% drift. For specimens of CFRPRW1 and CFRPRW2, there are no distinguishing difference observed.

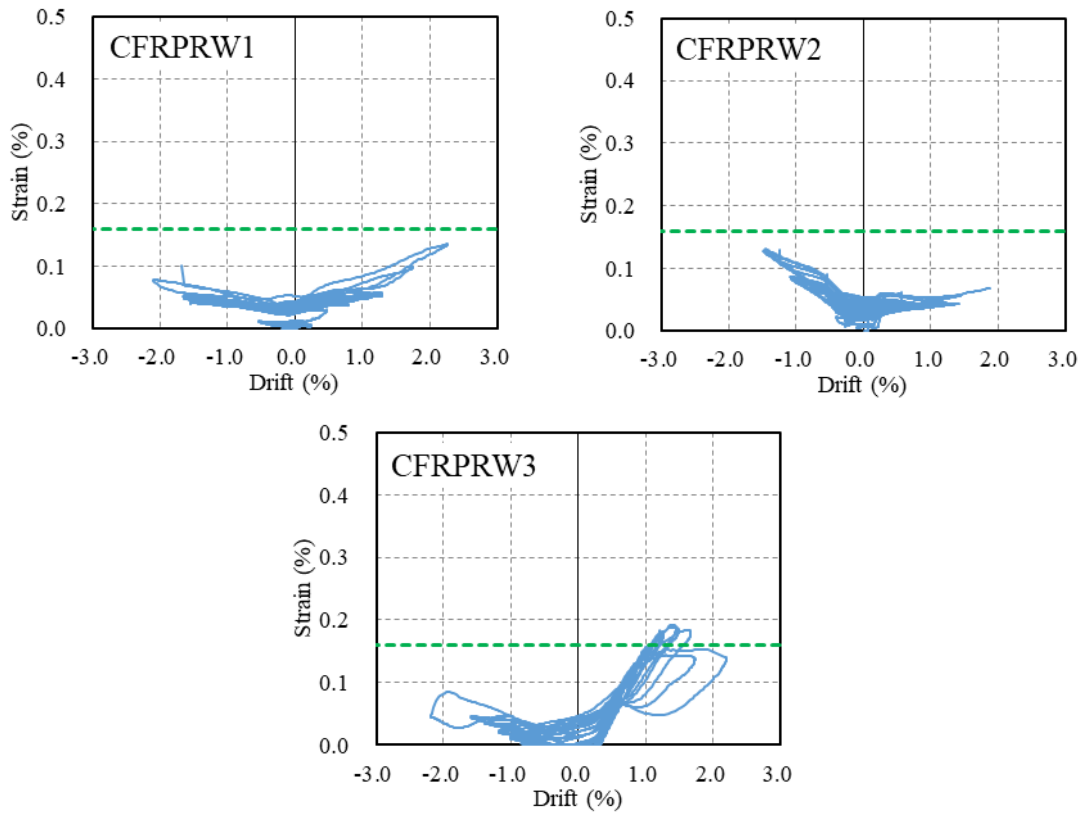


Fig. 2-20 Measured strains of horizontal web steel bars

2.3.5 Lateral deformation

The profile of average lateral displacement in push and pull directions along the wall height at several specific drifts is plotted in Fig. 2-21. The lateral displacement at each height within 600mm ~ 2000mm corresponds to the average of two DTs. The lateral displacement of the height, which varied from 600mm to 2560mm, increased linearly, while the slope of profile curve became sharp below the height of 600mm resulting in an inflection point for all specimens especially at large drifts. The inflection point of lateral displacement, which is noted using red solid circle in Fig. 2-21, reflects the rotation of concrete wall around the potential plastic hinge region within the approximate height of 600mm.

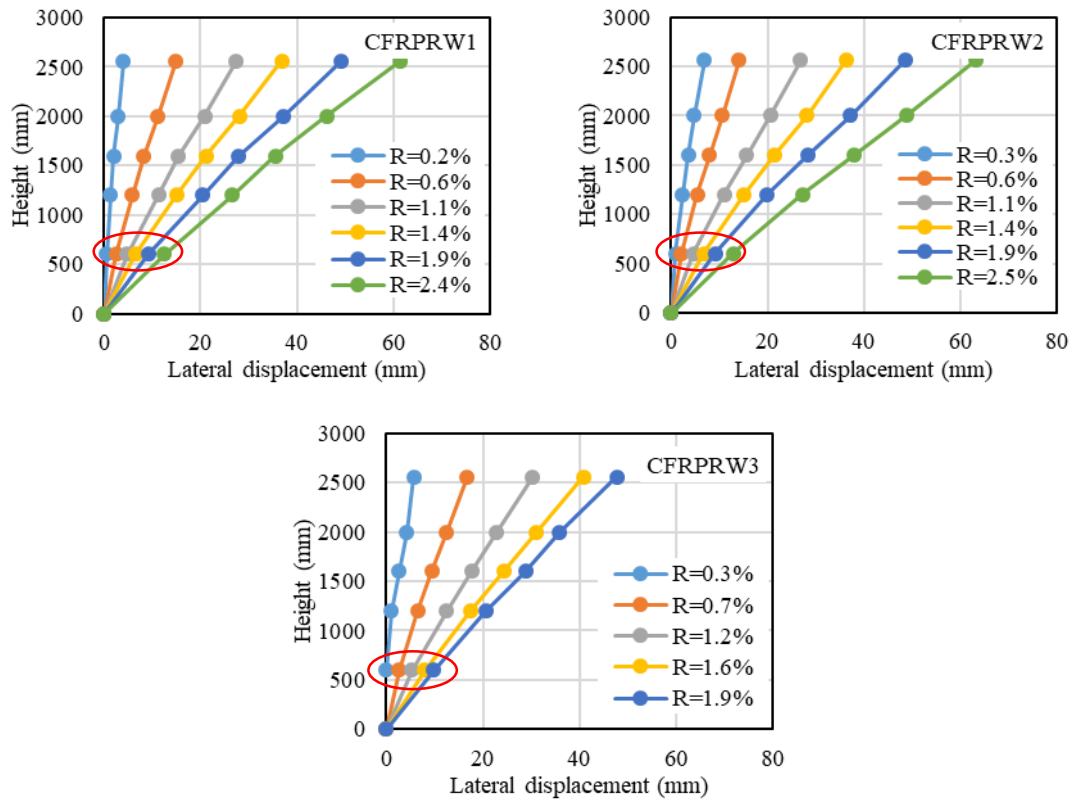


Fig. 2-21 Profile of the measured lateral displacement along the height of wall

2.3.6 Residual deformation and residual width of cracks

Residual deformation and residual crack width are two important parameters to assess the resilient capacity of concrete walls. Fig. 2-22 compares measured drifts of three specimens. Specimen CFRPRW1 only had 0.17% residual drift even unloading from the drift of 1.92%, exhibiting the 86% recovery of deformation after seismic loading. For CFRPRW2 and CFRPRW3 tested under relatively high axial load ratio, the excellent recovery of lateral drift reached 89% and 86%, respectively, after unloading from the maximum drift. The residual drifts after unloading from 1.5% drift of all specimens were less than 0.5% even under the high axial load ratio up to 0.33, indicating a repairable probability of 98% [2.3].

Comparison of the maximum crack width and residual crack width are shown in Fig. 2-23. The maximum crack width was measured at the peak lateral drift of each loading cycle, while the residual crack width was measured when the lateral load was unloaded to 0. The property of well recovery after cyclic loading similar to deformation recovering

can also be seen on cracks. The flexural crack of specimen CFRPRW1 was effectively reduced to only 23% of that measured at the corresponding peak drift level. Specimens remained well crack recovering capability though increasing the axial load ratio higher than 0.17.

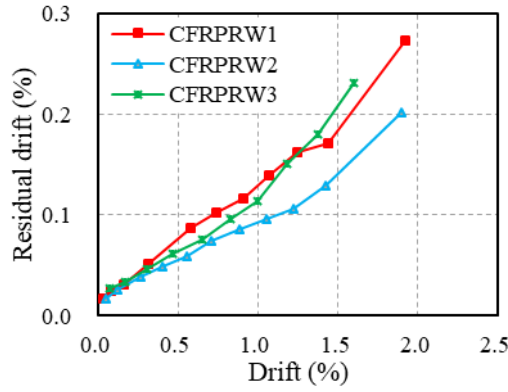


Fig. 2-22 Comparison of residual drifts

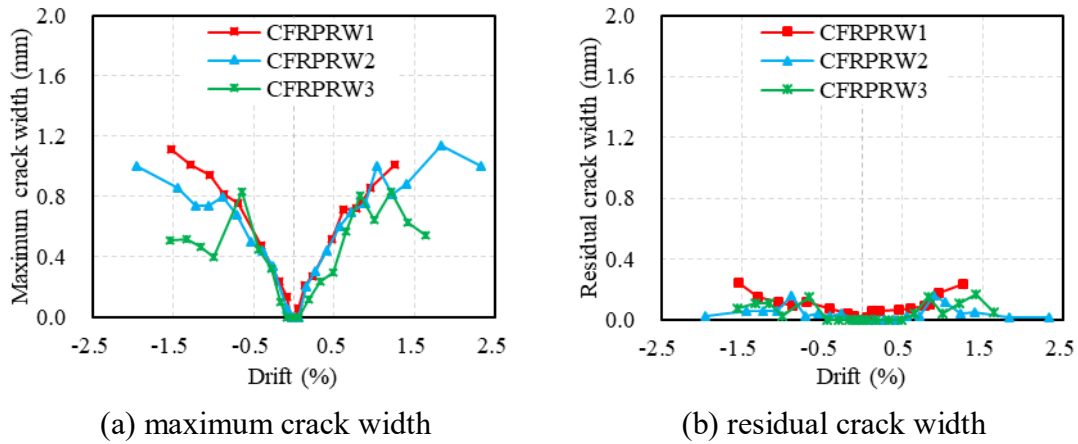


Fig. 2-23 Comparison of crack widths

2.3.7 Energy dissipation capacity

The energy dissipation capacity of seismic components can be assessed using the index of equivalent viscous damping (EVD). According to JGJ/T 101-2015 [2.2], the equivalent viscous damping can be calculated in form of

$$\zeta_{eq} = \frac{1}{2\pi} \cdot \frac{S_{(ABC+CDA)}}{S_{(OBE+ODF)}} \quad (2.8)$$

Where, $S_{(ABC+CDA)}$ is the area of shadow region encircled by the hysteretic loop, $S_{(OBE+ODF)}$ is the areas of two triangles OBE and ODF as shown in Fig. 2-24.

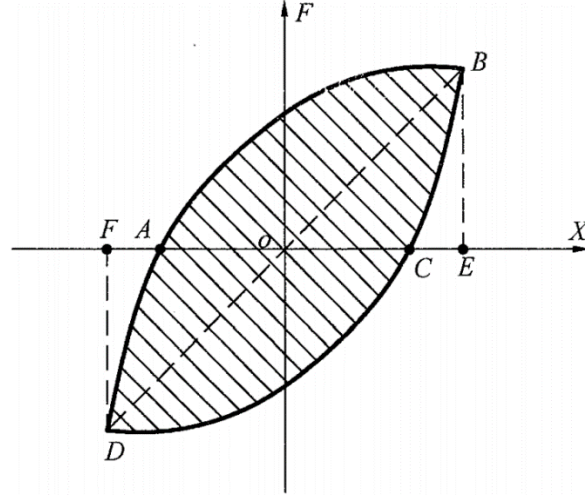


Fig. 2-24 Concept of the equivalent viscous damping [2.2]

The calculating results of EVD are shown in Fig. 2-25. The EVDs of all specimens exhibit an inverted triangle trend that EVDs declined first and remained the constant value of 0.1 within 0.5% ~1.0% drifts, after which increased with the drift. There is no evident difference on EVDs of three specimens before the drift of 1.0%, while the EVD of specimen under high axial load ratio went up faster than that under low axial load ratio after 1.0% drift. The measured EVD for the three test walls was beyond 0.095, implying that concrete walls reinforced by CFRP bars had higher energy-dissipation capacity than UPTP walls whose EVD was less than 0.04 [2.4].

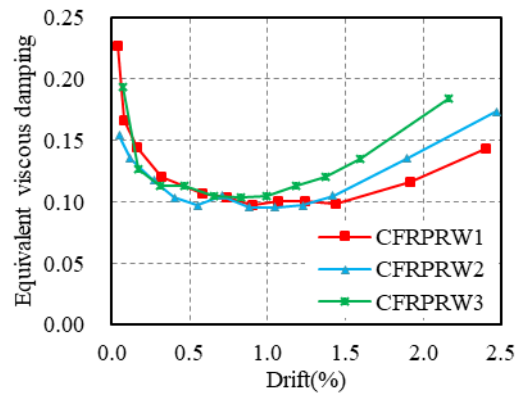


Fig. 2-25 Comparison of the equivalent viscous damping

2.4 Conclusions

In this chapter, three full-scale CFRP bar reinforcing concrete walls with rectilinear hoops in the boundary elements were fabricated and tested under cyclic reversed lateral load and axial compression with three axial load ratios of 0.17, 0.26 and 0.33 to investigate the seismic behavior of the walls under high axial load level. When supported by concrete walls with a span of 8.0 m in the transverse direction, the axial load ratios of 0.17, 0.25, and 0.33 approximately correspond to the total gravity sustained by inner walls at the lowest story of the buildings of 10-story, 15-story, and 20-story, respectively.

From the experimental results and discussion described in this chapter, the following conclusions can be drawn:

- 1) The ultimate state (peak point at the V-R curve) of concrete walls reinforced by CFRP bars was dominated by the rupture of the CFRP bars under compression. With the increment of axial load ratio, the stable drift-hardening capacity of concrete walls decreased from 2.0% drift to 1.2% drift due to the compression failure of CFRP bars and the crushing of concrete shell of the walls.
- 2) The residual drift ratio of the wall was kept less than 0.3% after unloading from the 1.5% drift level regardless of axial load level due to the long elasticity of CFRP bars. However, to ensure the walls a drift-hardening capacity of 2.0%, which has been defined as the drift of life safety for common concrete components in current design codes, the axial load ratio should be limited below 0.25 for concrete walls reinforced with CFRP bars.
- 3) The residual width of cracks decreased along with the increase of axial load and exhibited very small values. The recovery ratio of the residual crack width to corresponding maximum crack width measured at the drift of 1.5% reached 77%, 94%, and 88% in the specimens with axial load ratio of 0.17, 0.26, and 0.33, respectively.
- 4) The measured minimum equivalent viscous damping for the three test walls was

beyond 0.095, implying that concrete walls reinforced by CFRP bars had higher energy-dissipation capacity than UPTP walls.

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CHAPTER THREE

3. Seismic behavior of concrete walls reinforced by CFRP bars with boundary elements confined by circular hoops

3.1 Introduction

The experimental results of concrete walls reinforced by CFRP bars with boundary elements confined by rectilinear hoops described in chapter two have shown satisfactory drift-hardening capacity even under high axial load. However, rectilinear hoops had been placed in a composite form (perimeter hoop plus overlapped hoops or cross ties) to guarantee more effective confinement to the concrete core by mitigating the so-called arch effect as shown in Fig. 3-1 [3.1], which tends to cause some construction difficulty. Besides, according to the study by Blume and Newmark [3.2], the confining effect of rectilinear hoops is approximately 50% of that by circular hoops when the same amount of hoops is used.

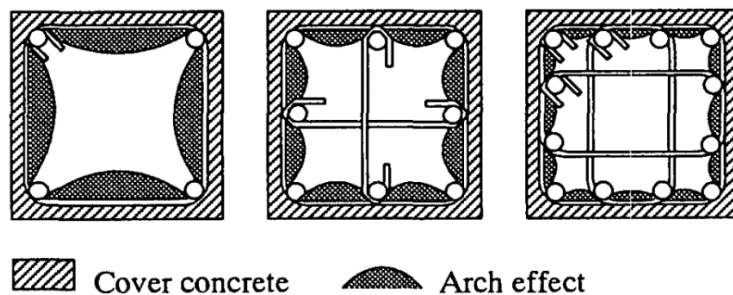


Fig. 3-1 Arch effect in the cross section [3.1]

From the above-mentioned observations, it is presumed that using circular hoops to confine the concrete in the boundary elements may provide more convenient and higher

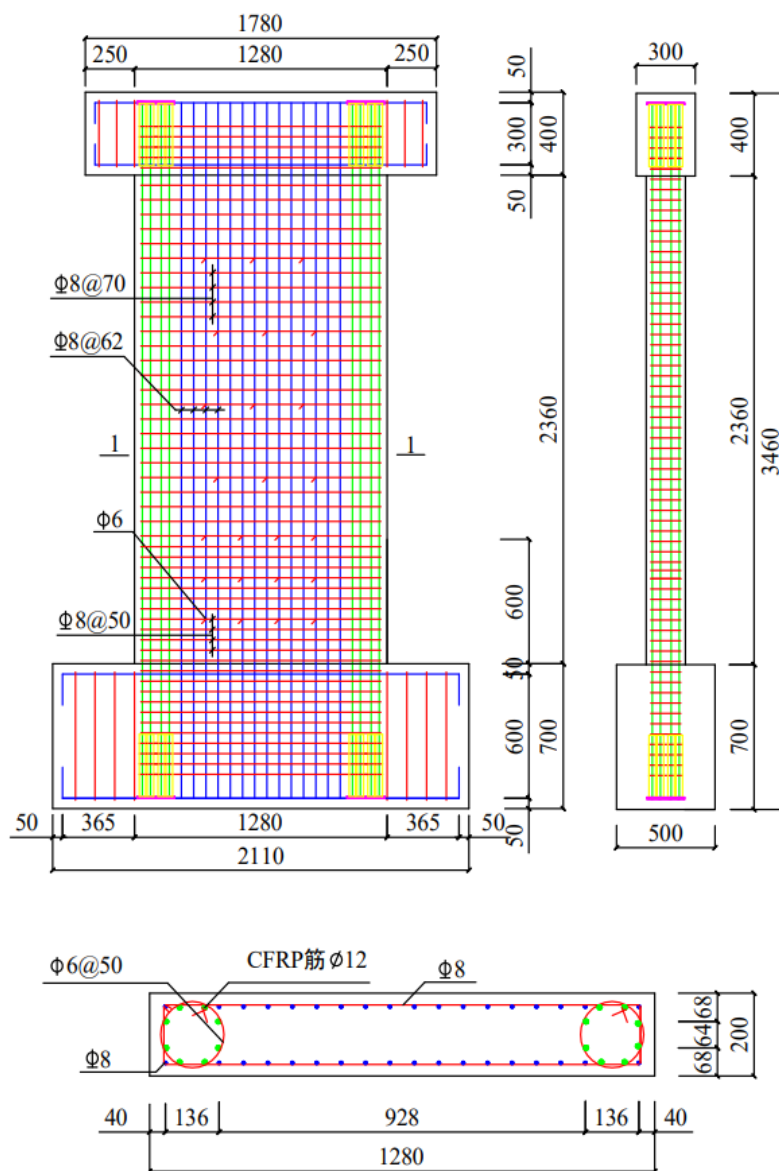
Chapter 3

confinement effect than composite rectilinear hoops, resulting in identical seismic performance of concrete walls with CFRP bars with much smaller amount of transverse hoops.

Objectives of this chapter are, 1) to present experimental information on the seismic behavior of concrete walls reinforced by CFRP bars with the boundary elements being confined by circular hoops, and 2) to investigate effects of circular hoops in the boundary elements on the seismic properties of the walls.

3.2 Experimental program

To achieve the research objectives, three full-scale specimens having the same geometry and reinforcement details as specimens in chapter two except for hoops in boundary elements were fabricated and tested. Because of materials properties, loading apparatus and instrumentation of the experimental program in this chapter were same as that in chapter two, only the details of test walls will be described in this section.



Section 1-1

Fig. 3-2 Dimensions and reinforcement details of test walls

Fig. 3-2 and Table 3-1 shows the reinforcement details and outlines of test specimens, respectively. All specimens with the same geometry were reinforced by the same arrangement and tested under three different axial load ratios of 0.17, 0.26 and 0.33. It can be seen that specimens with circular hoops have the identical CFRP bars in boundary elements with the specimens having rectilinear hoops and each end of CFRP bars was also anchored using the same method as shown in Fig. 2-2 in last chapter. The circular hoops were placed in boundary elements at the spacing of 50mm leading to the volumetric ratio to the confined core of 1.4%, which is only 36% of the volumetric ratio of rectilinear hoops at the same spacing. The web vertical and horizontal reinforcement was uniform as specimens described in previous chapter.

The experimental variable was also the axial load ratio, which varied from 0.17 to 0.33. Specimen notations were marked as CFRPHW1, CFRPHW2 and CFRPHW3 to correspond to axial load ratios of 0.17, 0.26 and 0.33, respectively.

Table 3-1 Outlines of test specimens

Notati on	L/l_w	n	Boundary element		Web	
			Longitudi nal bars	Hoops	Vertical bars	Horizontal bars
CFRP HW1	2	0.17	8-CFRP bars	HPB300 @50 $\rho_h=1.4\%$	HRB335 @62 $\rho_v=0.8\%$	Region above the height of 600mm: HRB335@70 $\rho_w=0.7\%$
CFRP HW2		0.26	4-HRB335 bars			Region below the height of 600mm: HRB335@50 $\rho_w=1.0\%$
CFRP HW3		0.33				

Notes: L/l_w , shear span ratio; n , axial load ratio, $n=N/(Af'_c)$; N , axial load; A , sectional area of wall; f'_c , axial compressive strength of concrete; ρ_h , volumetric ratio of hoops to the confined core measured center to center of perimeter hoop of boundary element; ρ_v , steel ratio of vertical web reinforcement; ρ_w , steel ratio of horizontal web reinforcement.

3.3 Test results and observation

3.3.1 Observed behavior

The crack patterns of specimens are shown in Fig. 3-3 through Fig. 3-5. The drift below each crack pattern is the targeted one of loading program shown in Fig. 2-6, while the drift in bracket behind loading targeted drift represent the actual drift measured by DT. The last pattern of each figure shows the cracks observed at the end of test. Cracks observed in push and pull directions are represented using red and blue lines, respectively, and the spalling of concrete is shown in black solid. Besides, the dotted-line grids with the size of 100mm are utilized to give a reference to the crack pattern.

For specimen CFRPHW1, when the lateral drift reached 0.06% in pull direction, the flexural crack was first observed at the section 370mm away from the wall base. The first flexural crack in push direction arose at the drift of 0.19%. As increment of lateral drift up to 0.31%, flexure-shear cracks were observed. The cover concrete began to spall off as the drift increased to 0.90%. Cracks propagation distributed along the height of 1900mm after experiencing the final loading cycle with the targeted drift of 3.0%.

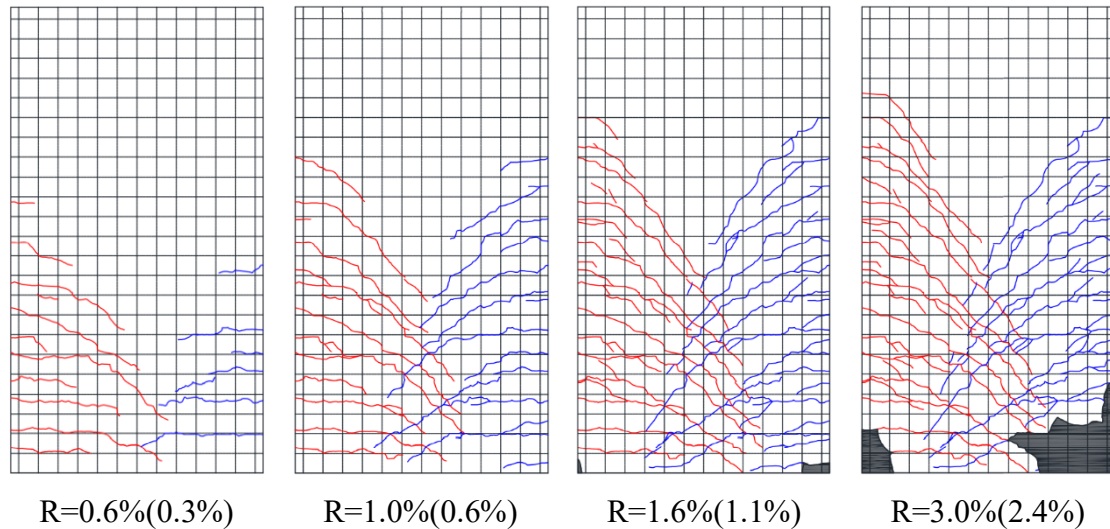


Fig. 3-3 Crack patterns of specimen CFRPHW1

The flexural crack was also first observed during the test of specimen CFRPHW2 at the drift of 0.07%. The next specific observed behavior was the flexure-shear crack when

increasing the drift up to 0.13%. As reversed cyclic lateral loading continued, spalling of cover concrete started to appear at 1.04% drift. At the end of test, cracks propagated up to the section of 1730mm away from the base stub.

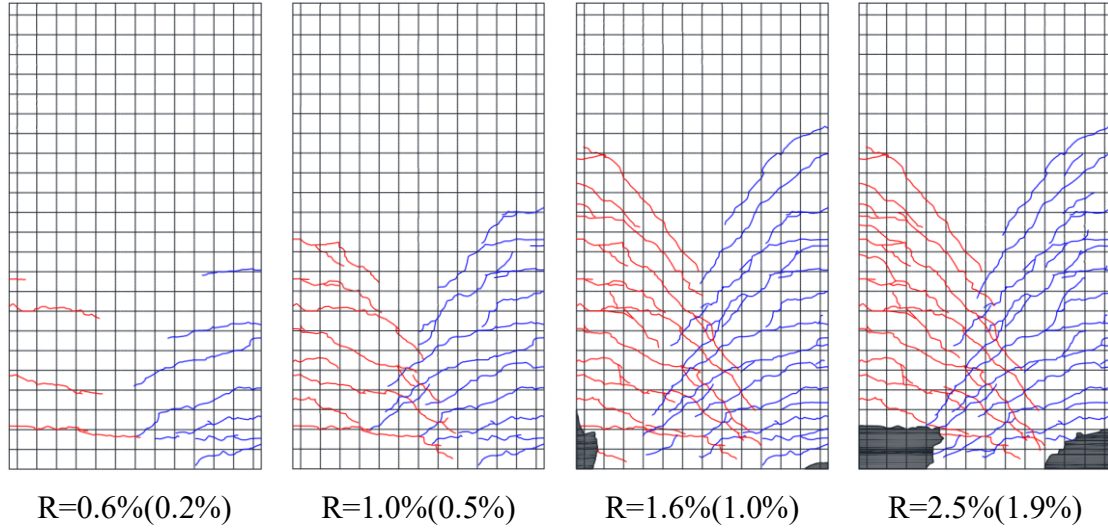


Fig. 3-4 Crack patterns of specimen CFRPHW2

For specimen CFRPHW3, the flexural and flexure-shear cracks were observed at the drifts of 0.05% and 0.12%, respectively. When the drift increased to 0.56%, the concrete spalling was observed. As shown in Fig. 3-5, the cracks propagating height reached up to 1600mm accounting for 68% of the overall wall panel.

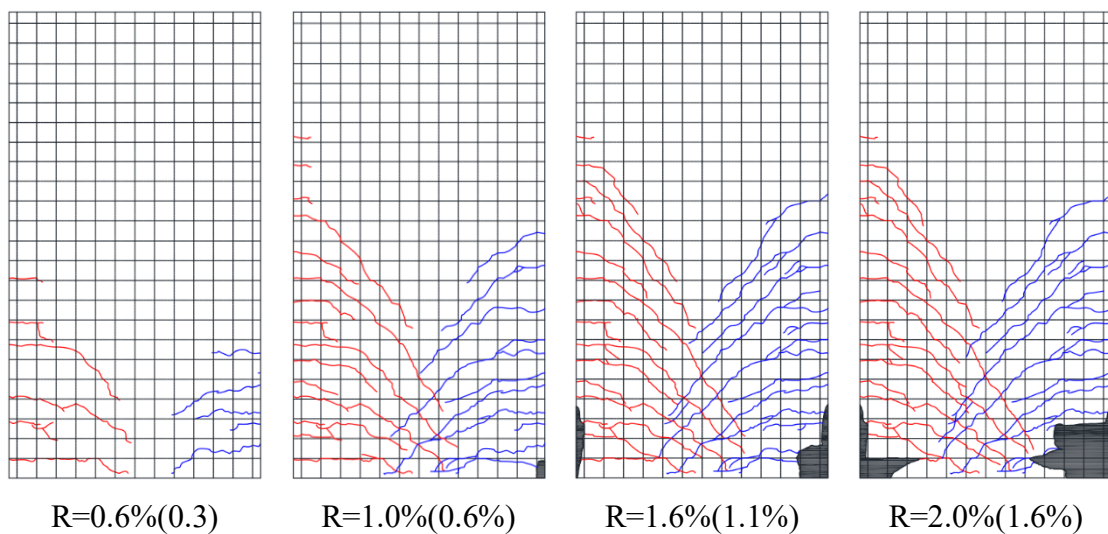


Fig. 3-5 Crack patterns of specimen CFRPHW3

The crack propagating height declined as increasing the axial load ratio comparing crack patterns of three specimens, which indicates the restraining effect of axial load on cracks propagation.

3.3.2 Lateral force-drift responses

Fig. 3-6 shows the V-R curves of specimens with circular hoops, where the point of maximum lateral load was covered using the solid red circle. Besides, the P- Δ effect on lateral force was represented by the dotted lines. Table 3-2 summarizes primary experimental results.

The CFRP bars reinforcing concrete walls with boundary elements confined by circular hoops can also exhibit well resilience with little residual deformation up to 1.5% drift even under the high axial load ratio of 0.33 as shown in Fig. 3-6. For CFRPHW1 and CFRPHW2 under relatively lower axial load ratios than 0.33, they can maintain resilient capacity up to near 2.0%. The compression failure of CFRP bars and crushing of concrete shell account for the decline of lateral force that resulted in failure of walls.

The effects of axial load ratio can be found by comparison of three specimens in Table 3-2 and Fig. 3-7. The method to obtain envelop curves is identical with specimens with rectilinear hoops, which has been described in detail already. As shown in Fig. 3-6 and Table 3-2, The maximum lateral load increased with the increment of axial load ratio. The specimen CFRPHW3 had the highest maximum lateral load of 926kN that was 8% and 13% larger than CFRPHW2 and CFRPHW1, respectively. While the corresponding drift to maximum lateral load showed the inverse tendency that the drift decreased from 1.64% to 1.02% as increasing the axial load ratio.

Table 3-2 Primary results of specimens

Notation	n	V_{max} (kN)	R_{max} (%)	V_{min} (kN)	R_{min} (%)	V_{ave} (kN)	R_{ave} (%)
CFRPHW1	0.17	846.96	1.86	-790.97	-1.42	818.97	1.64
CFRPHW2	0.26	859.59	1.08	-854.98	-1.38	857.29	1.23
CFRPHW3	0.33	950.91	1.16	-902.94	-0.88	926.93	1.02

Notes: n , axial load ratio; V_{max} , peak lateral force in push direction; R_{max} , corresponding drift to V_{max} ; V_{min} , peak lateral force in pull direction; R_{min} , corresponding drift to V_{min} ;

V_{ave} , average of V_{max} and V_{min} ; R_{ave} , average of R_{max} and R_{min} .

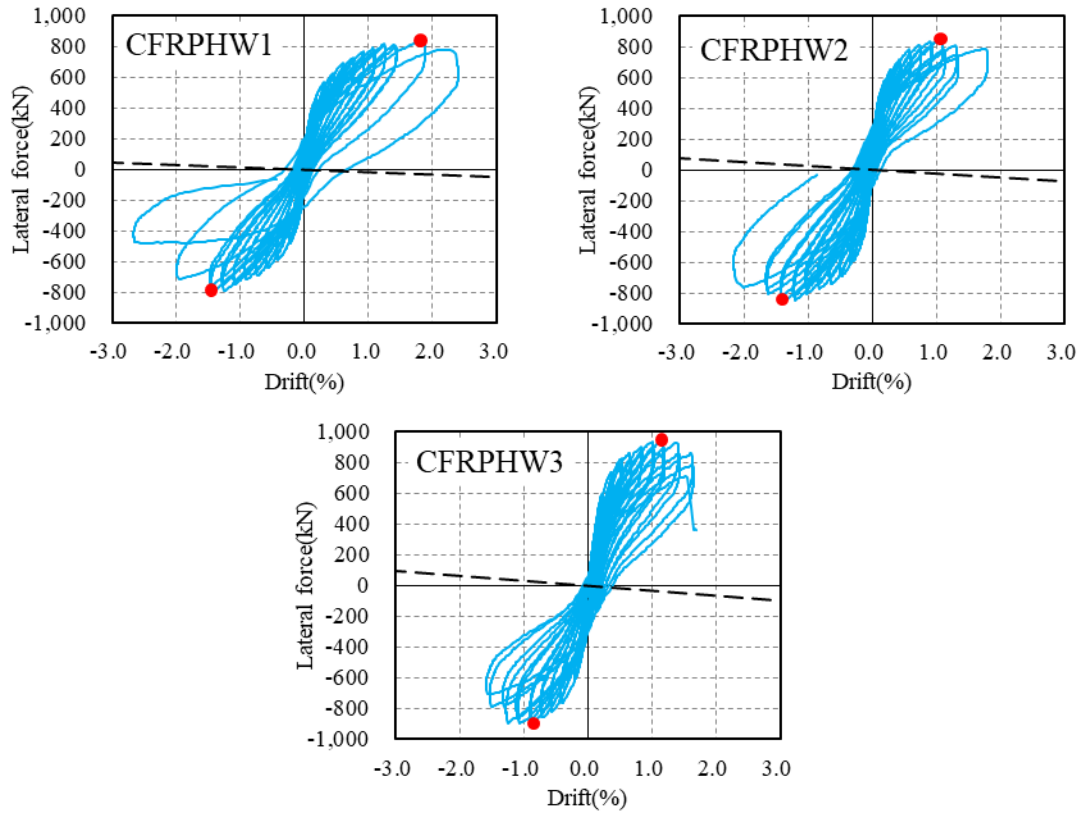


Fig. 3-6 Measured lateral force-drift (V-R) relationships of specimens

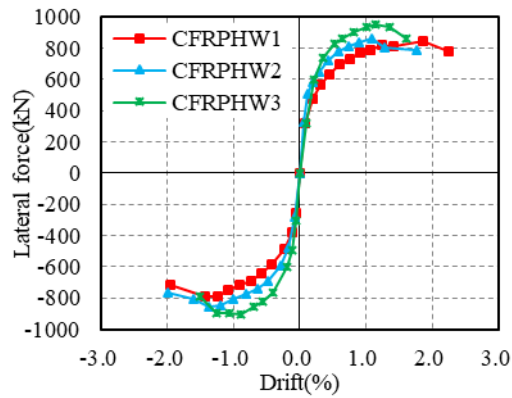


Fig. 3-7 Comparison of envelop curves

3.3.3 Stiffness Degradation

The experimental secant stiffness of each drift level was calculated according to the Eq. (2.2) and calculating initial stiffness was obtained by the Eq. (2.3) through Eq. (2.6). All

these equations can be found in the last chapter.

Table 3-3 compares the experimental initial stiffness (measured secant stiffness at the first loading cycle) and calculating initial stiffness of circular hoops confining walls. It can be seen that the calculating initial stiffness agreed well with the experimental initial stiffness with the difference range of 4% ~13%.

Fig. 3-8 shows comparison of the stiffness degradation caused by cumulative damage under cyclic loading. The specimen CFRPHW1 exhibited nearly same stiffness degradation curve as CFRPHW2. For specimen CFRPHW3, its secant stiffness was larger than other two specimens at same specific drifts.

Table 3-3 Comparison between experimental and calculating initial stiffness

Notation	Experiment	Calculation				K_1/K_e
	K_1 (kN/mm)	K_e (kN/mm)	Δ_B (mm)	Δ_S (mm)	Δ_S/Δ	
CFRPHW1	173.84	199.36	4.17	0.84	0.17	0.87
CFRPHW2	190.43					0.96
CFRPHW3	173.87					0.87

Notes: K_1 , experimental initial stiffness; K_e , calculating initial stiffness; Δ_B , lateral displacement caused by flexural deformation under the lateral loading of 1N; Δ_S , lateral displacement caused by shear deformation under the lateral loading of 1N; Δ , $\Delta_B + \Delta_S$.

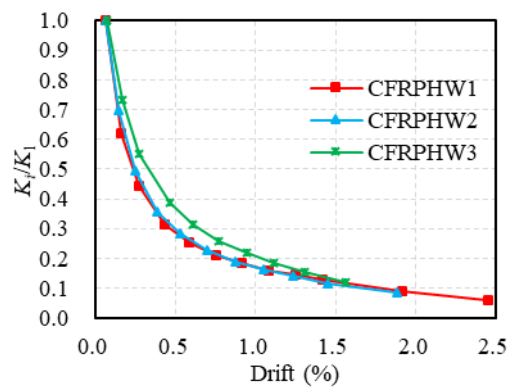


Fig. 3-8 Comparison of the Stiffness degradation

3.3.4 Strains measured in steel bars and CFRP bars

Fig. 3-9 shows the measured strain of CFRP bar at the height of 100mm for CFRPHW3, while for specimens CFRPHW1 and CFRPHW2, the CFRP strain at the height of 300 mm is shown because strain gauges broke down. The ultimate tensile strain of the CFRP bar is also plotted by green dotted line. Tensile rupture of CFRP bars was not observed for all specimens in spite of the axial load ratio according to measured strains of CFRP bars. It is reasonable to infer that the compression failure of CFRP bars occurred under high compression stress, which account for the decline of lateral resistance of walls. The other point obtained from strain-drift curves of CFRP bars is the origin-oriented property resulting from ultra-high tensile strength with linear elasticity of CFRP bars, which offers components resilience.

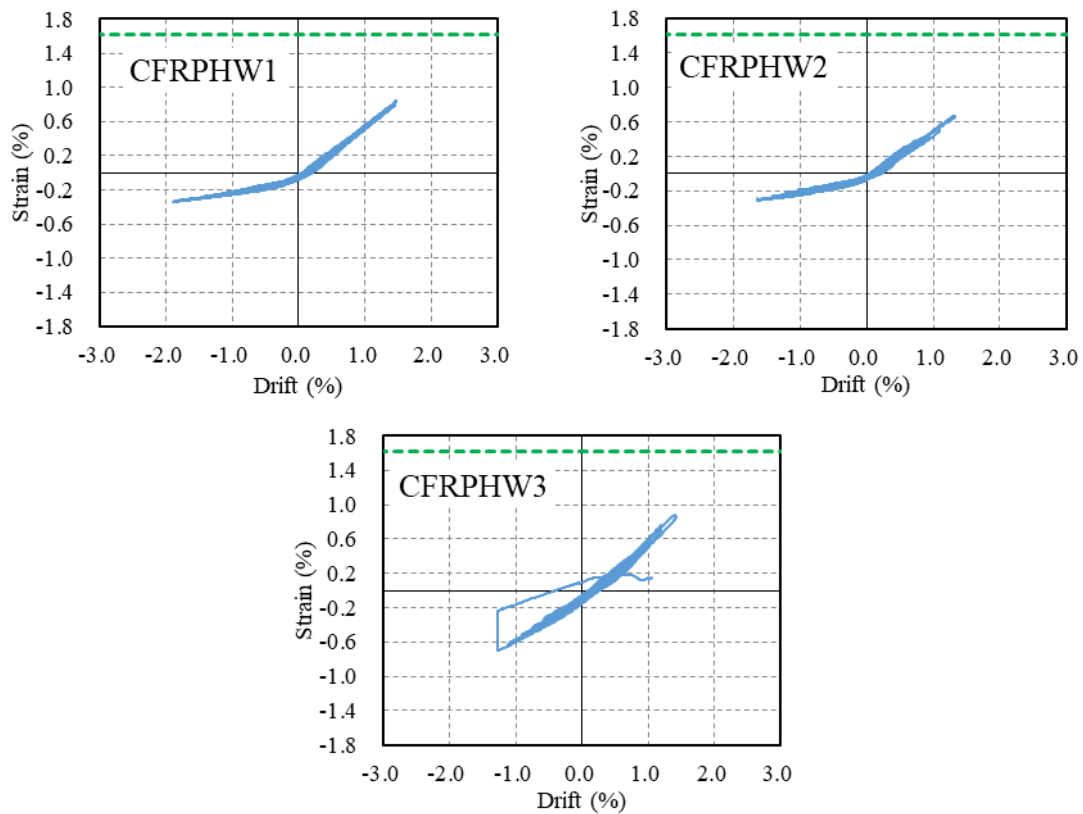


Fig. 3-9 Measured strains of CFRP bars

The strain profiles of CFRP bars along height of walls are shown in Fig. 3-10. The slippage of CFRP bars within the potential hinge region can also be observed as specimens with rectilinear hoops in spite of the axial load ratio.

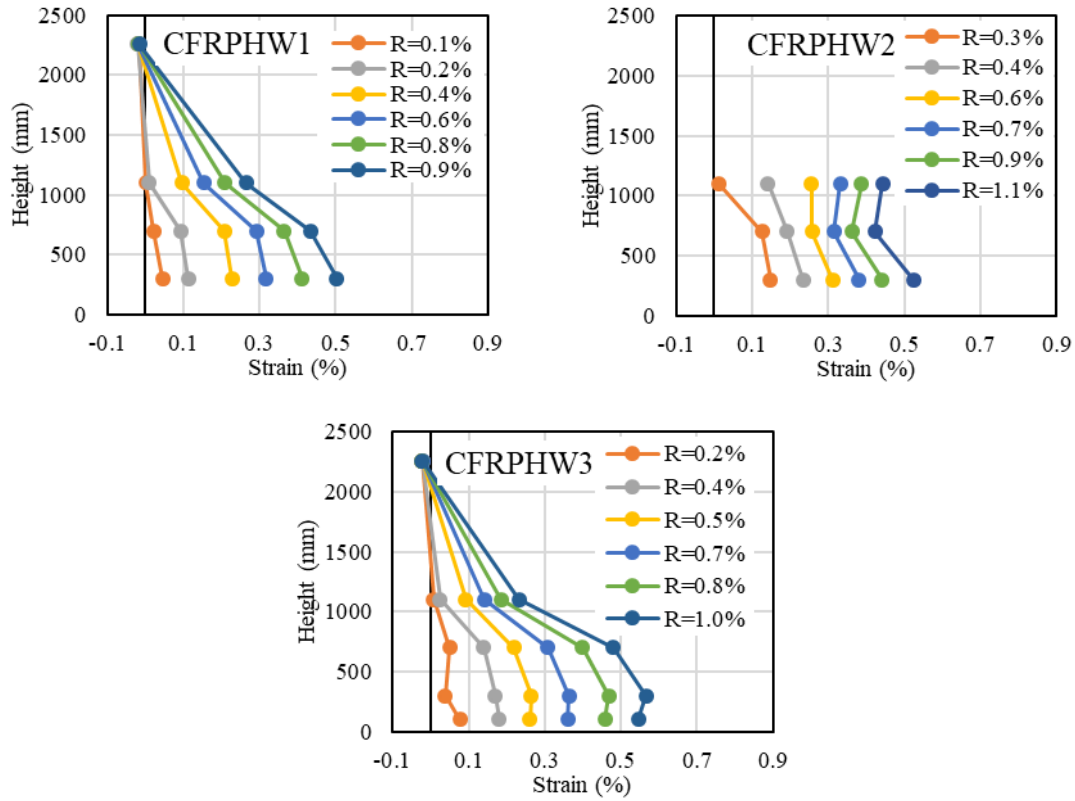


Fig. 3-10 Strains distribution of CFRP bar

Strains of longitudinal steel bars in boundary elements at the height of 100 mm are measured and shown in Fig. 3-11, in which green dotted lines represent yield strain of the steel bars. The longitudinal steel bars of all specimens yielded before the drift of 0.5% resulting large plastic deformation under the reversed load.

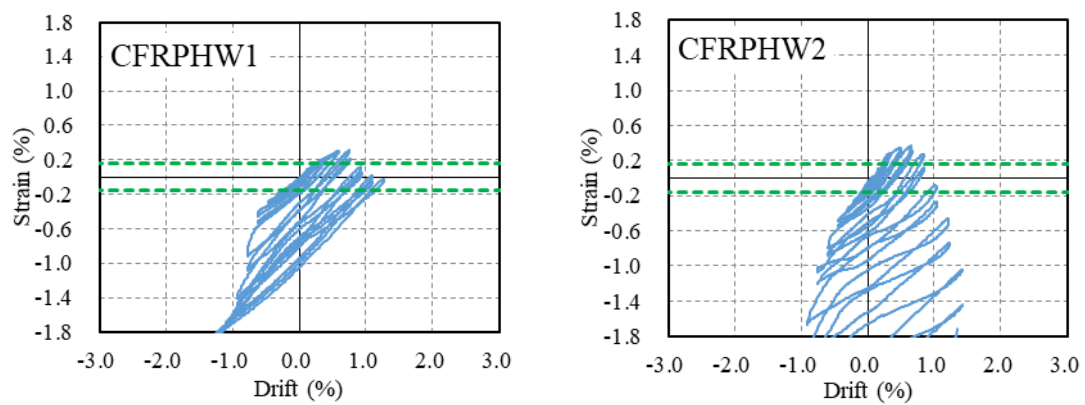


Fig. 3-11 Measured strains of longitudinal steel bars

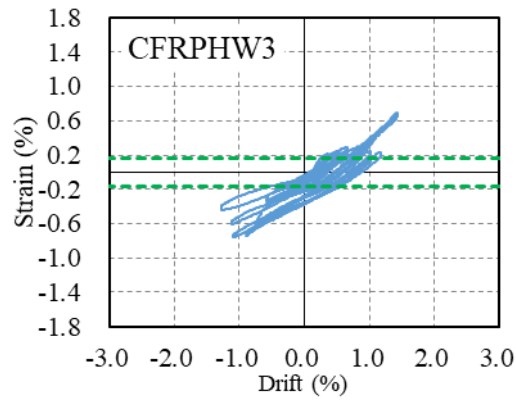


Fig. 3-11 Continued

Fig. 3-12 shows measured strains of circular hoops at the height of 100mm. The yield strain of hoop is also shown as green dotted line in each figure. Circular hoops yielded under the axial load ratios of 0.26 and 0.33 at the drift of 1.2%. For specimen CFRPHW1 tested under 0.17 axial load ratio, yield of the circular hoop did not be observed according to the strain-drift relationship.

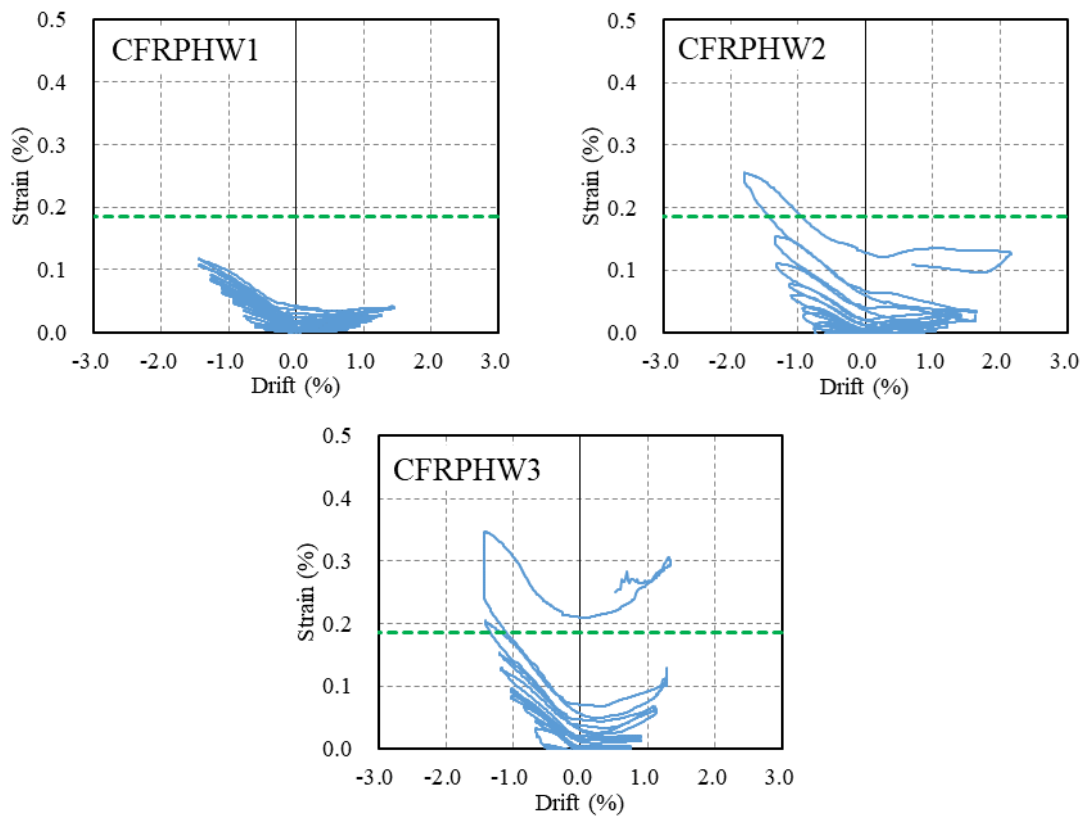


Fig. 3-12 Measured strains of hoops in boundary elements

The horizontal web steel bar placed in each specimen at the height of 100 mm did not yield during the test as shown in Fig. 3-13.

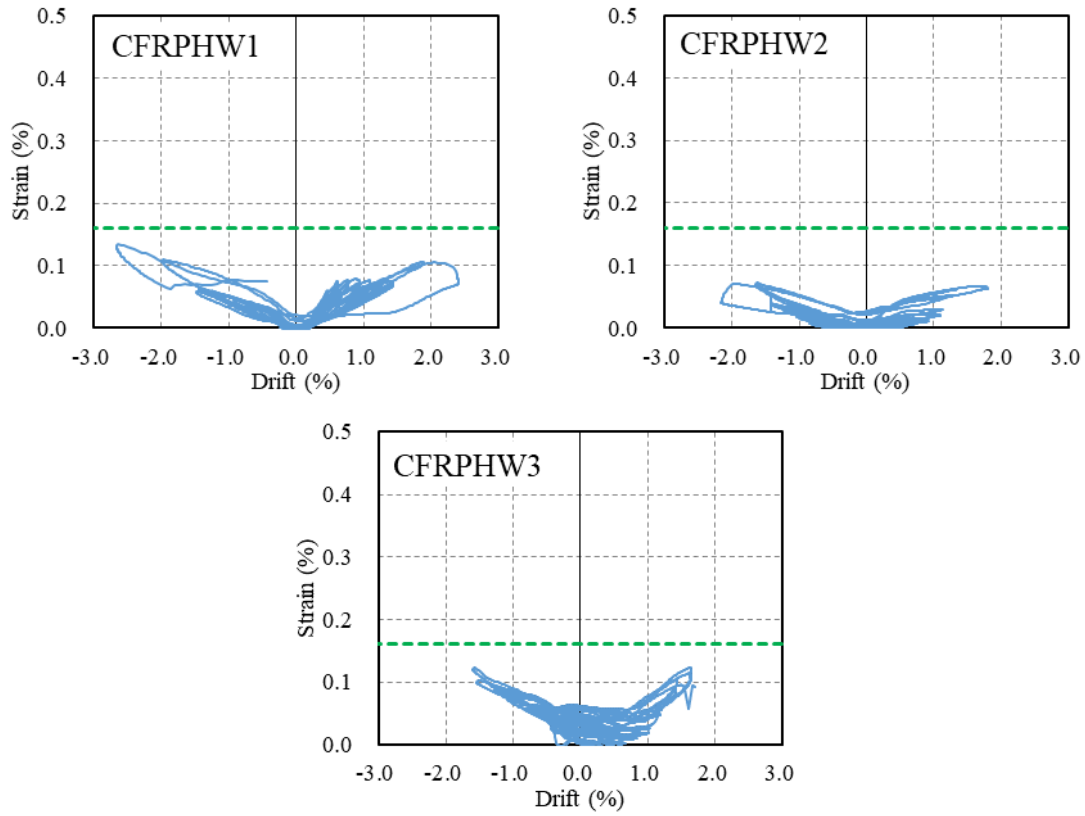


Fig. 3-13 Measured strains of horizontal web steel bars

3.3.5 Lateral deformation

Lateral displacements of the heights of 600mm, 1200mm, 1600mm, 2000mm and 2560mm at several drifts are plotted in the form of profile in Fig. 3-14 to represent deforming characteristic of specimens. The red solid ellipses mark the inflection point of the profile of lateral displacement that means large rotation occurred below this point resulting in the variation on slope of the profile.

3.3.6 Residual deformation and residual width of cracks

Residual deformation and residual crack width of concrete walls with circular hoops are shown in Fig. 3-15 and Fig. 3-16, respectively. All specimens exhibited well recovering capacity on deformation even under the axial load ratio up to 0.33. The recovering ratio

of residual drift to peak drift at the last loading cycle reached over 85% for all specimens.

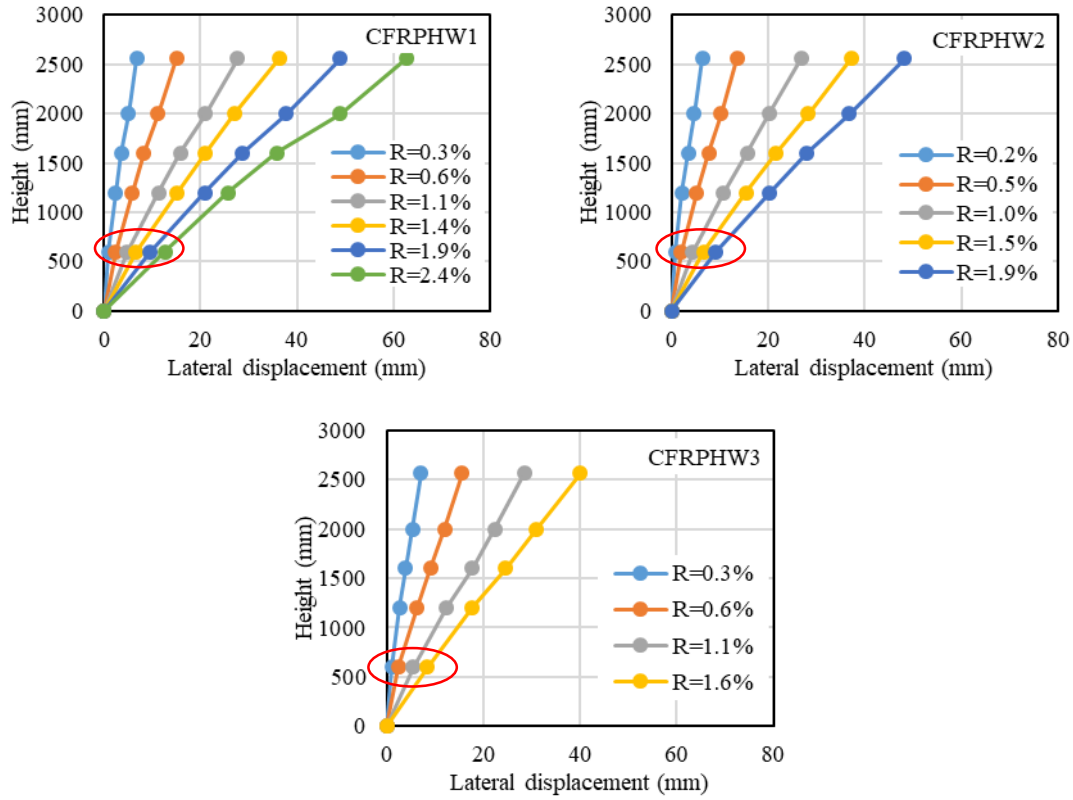


Fig. 3-14 Profile of the measured lateral displacement along the height of wall

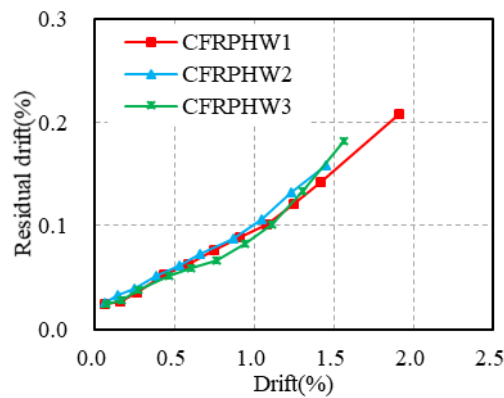


Fig. 3-15 Comparison of the measured residual drifts

Fine recoverability can also be observed from the residual crack width. As shown in Fig. 3-16, the ratio of residual crack width to corresponding maximum crack width at the ultimate drift is only 25% for the specimen CFRPHW1, and that ratio still remains low values of 7% and 15% for other two specimens under higher axial load ratios of 0.26 and

0.33, respectively.

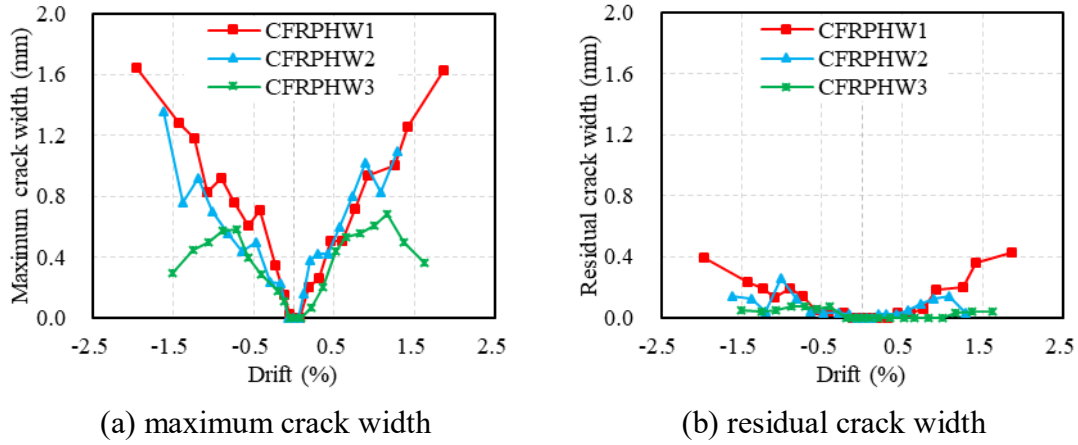


Fig. 3-16 Comparison of the crack widths

3.3.7 Energy dissipation capacity

The equivalent viscous damping (EVD) is utilized to assess the energy dissipation capacity of concrete walls with circular hoops as well. Fig. 3-17 shows comparison of the EVD. Three walls exhibited almost the same EVD of about 0.1 within 0.5% ~1.0% drifts. When the drift beyond 1.0%, The EVDs of CFRPHW2 and CFRPHW3 remained uniform and increased with drift until failure, while the EVD of CFRPHW1 was slightly lower and reached up to 0.2 at the drift of 2.5%. Like the walls described in chapter two, the measured minimum EVD for the test walls confined by circular hoops was 0.095 that was larger than the maximum EVD (=0.04) of UPTP walls, indicating higher energy-dissipation capacity than UPTP walls [3.3].

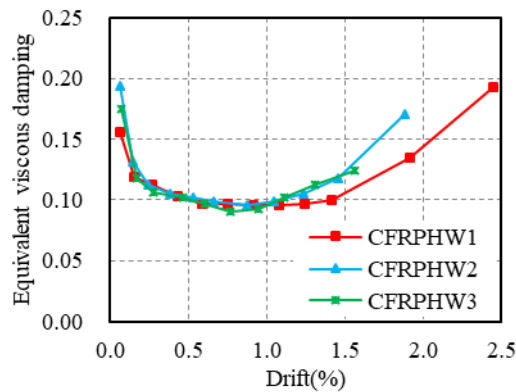
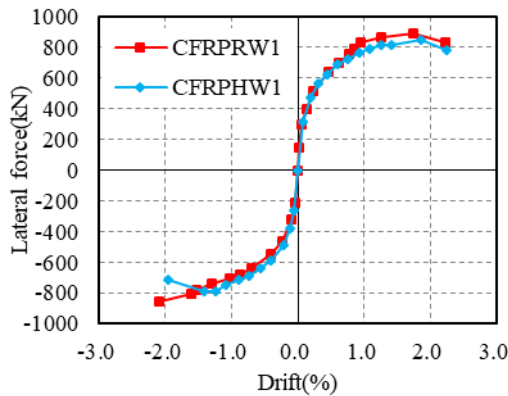


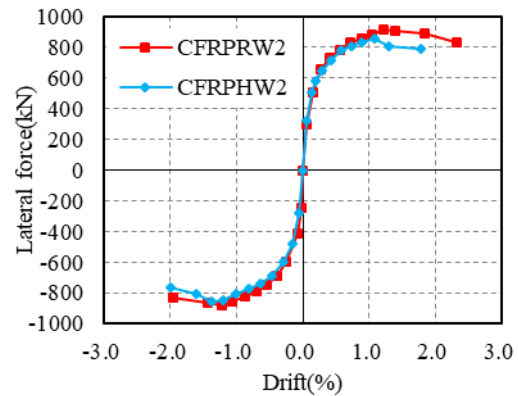
Fig. 3-17 Comparison of the equivalent viscous damping

3.4 Comparisons with specimens confined by rectilinear hoops

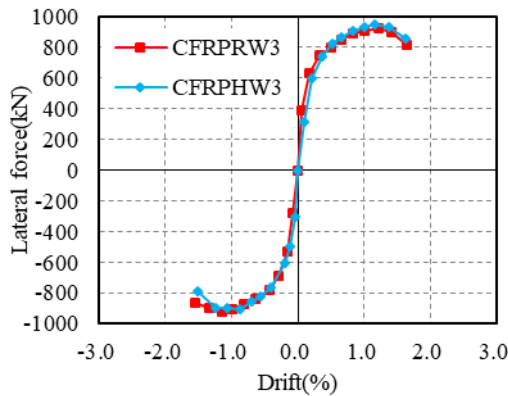
To clearly understand the effect of hoops in boundary elements, the envelop curves of specimens with circular hoops are compared with that of specimens confined by rectilinear hoops as shown in Fig. 3-18. In general, envelop curves of circular-hoop series overlap that of rectilinear-hoop series before the drift of 1.0%, and the decline of lateral force at large drift is more evident for circular-series in spite of the axial load ratio. For the axial load ratio of 0.17, rectilinear-hoop confining walls exhibited the slightly larger lateral force after 1.0% drift and ultimate drift in pull direction than circular-hoop confining one. The similar situation can be observed for the axial load ratio of 0.26 that CFRPRW2 had the ultimate drift over 2.0%, while the ultimate drift of CFRPHW2 did not exceed 1.8% in push direction. When the axial load ratio reached 0.33, two series of specimens exhibited the same deformability.



(a) axial load ratio of 0.17



(b) axial ratio of 0.26



(c) axial load ratio of 0.33

Fig. 3-18 Effect of hoops in boundary elements

3.5 Conclusions

In order to investigate if confinement of concrete in the boundary elements by circular hoops can ensure sufficient drift-hardening capacity to concrete walls reinforced with CFRP bars, three full-scale specimens were fabricated and tested under the same experimental condition as described in chapter two. From the experimental results and comparisons, the following conclusions can be drawn:

- 1) While the steel amount of circular hoops (volumetric ratio of 1.4%) was only 36% of that of the composite rectangular hoops used in the boundary element of the walls described in chapter two, concrete walls reinforced by CFRP bars with boundary elements being confined by circular hoops exhibited stable lateral resistance up to about 2.0% drift if the axial load ratio was less than 0.26. The specimen under high axial load with axial load ratio of 0.33 showed stable lateral resistance up to 1.2% drift. These facts imply high efficiency of confinement by circular hoops, use of which could simultaneously reduce the steel amount of transverse bars and improve construction of the walls.
- 2) The residual drift ratios of all specimens were kept less than 0.3% after unloading from the 1.5% drift level regardless of axial load level, indicating high reparability of CFRP bar reinforcing concrete walls.
- 3) High reparability was also observed from the residual crack width. The ratio of residual crack width to corresponding maximum crack width at the ultimate drift is only 25% for the wall with axial load ratio of 0.17, and that ratio remained as low as 7% and 15% for the walls under higher axial load with axial load ratios of 0.26 and 0.33, respectively.
- 4) Confinement by circular hoops couldn't change the ultimate state of concrete walls reinforced by CFRP bars, which still was dominated by the rupture of the CFRP bars under compression. With the increment of axial load ratio, the stable drift-hardening capacity of concrete walls decreased from 2.0% drift to 1.2% drift due to the compression failure of CFRP bars.

- 5) Like the walls described in chapter two, the measured minimum equivalent viscous damping for the three test walls confined by circular hoops was beyond 0.095, indicating higher energy-dissipation capacity than UPTP walls.

References

- [3.1] Sakino K., Sun Y., “Stress-strain curve of concrete confined by rectilinear hoop,” *Journal of Structural and Construction Engineering*, AIJ, No. 461, 1994, pp. 95-104.
- [3.2] Blume J. A., Newmark N. M., Corning L. H., “Design of multistory reinforced concrete buildings for earthquake motions,” Portland Cement Association, 1961.
- [3.3] Kurama Y., “Hybrid post-tensioned precast concrete walls for use in seismic regions,” *PCI Journal*, Vol. 47, No. 5, 2002, pp. 36-59.

CHAPTER FOUR

4. Numerical analytical methods to assess hysteretic behavior of concrete walls reinforced by CFRP bars

4.1 Introduction

As described in chapter two and chapter three, concrete walls reinforced by CFRP bars exhibited stable lateral resistance and well resilience, which can be attributed to the long linear elasticity of CFRP bar due to its ultra-high tensile strength and low bond strength. In particular, due to the low bond strength of CFRP bars, the well-known plane-remain-plane assumption, which has been generally made as conducting analysis of cyclic behavior of concrete components made of longitudinal deformed rebars with high bond strength, may not be applicable to the analysis of CFRP bar reinforcing concrete walls. A numerical analysis method is apparently desirable for evaluating seismic performance of concrete walls with CFRP bars.

Sun and his colleagues have proposed an integrated analytical method for concrete components, and the proposed method can take into account the effect of bond slippage of longitudinal rebars on the cyclic response of general concrete columns and walls reinforced by rebars with high and/or low bond strength [4.1-4.3].

Objective of this chapter are to develop analytical method to predict seismic performance of concrete walls reinforced with CFRP bars by modifying the analytical method proposed by Sun et al., and to verify reliability and accuracy of the numerical analytical method.

4.2 Numerical analytical method

4.2.1 Simplified analysis model for concrete wall

The simplified analysis model for concrete wall is shown in Fig. 4-1. Rotation of the wall concentrates in the plastic hinge with uniform curvature to simplify calculating procedures. Then the relation between curvature of cross section, ϕ , and the drift caused by flexural deformation, R_f , can be simply obtained as shown in Fig. 4-1. It is important to figure out the ratio of the lateral drift from flexural deformation to overall drift before analyzing, of which calculating methods will be described after.

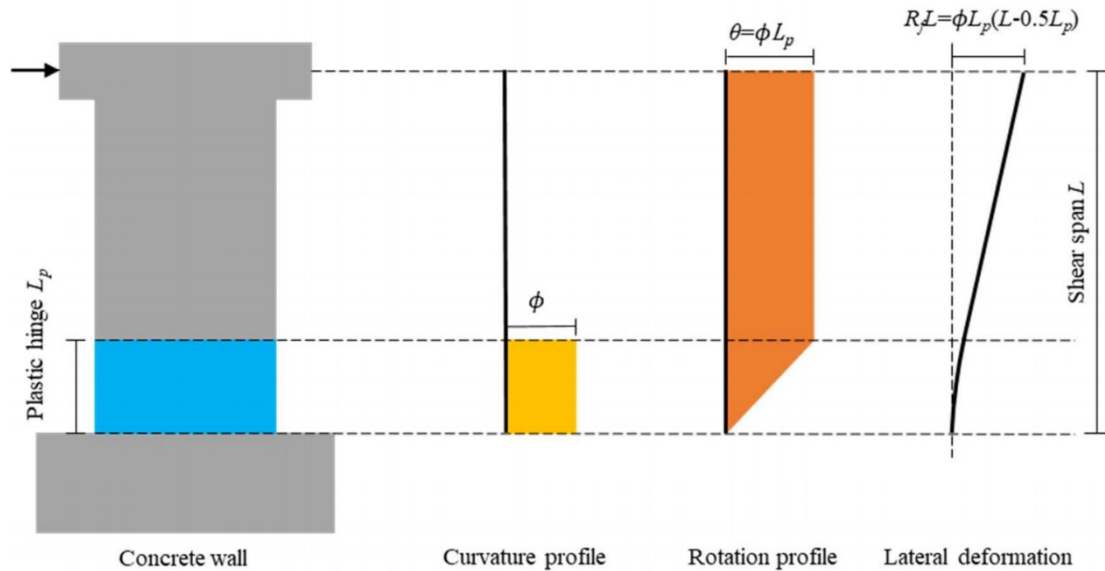


Fig. 4-1 Simplified analysis model for concrete wall

4.2.2 Bond-slip spring zone

To take into account the slippage of CFRP bars in concrete, CFRP bars are considered to connect with concrete through several bond-slip springs, which is called finite springs method (FSM). In the FSM, the concrete wall is divided into three zones as shown in Fig. 4-2. Bond-slip springs in the component side are shown as an example to explain the FSM method. As shown in Fig. 4-2, The bond-slip spring zone is divided into n segments, each of them is a bond-slip spring, through which the CFRP bar is connected with the concrete. The bond-slip relation is defined by the function of $\tau ()$ with variables of bond stress and

slippage. The objective of bond-slip springs model is to obtain stress of CFRP bar, σ_C , as the slip value, S_C , is given. That calculating procedure based on the model shown in Fig. 4-2 is summarized as follows:

- 1) Assign the S_C to S_0 that is the slip of the first segment, $S_C=S_0$.
- 2) Give an initial value of σ_0 to the stress of CFRP bar in the first segment.
- 3) The CFRP bar stress and slip in adjacent segment, σ_1 and S_1 can be calculated by Eq. (4.1) and Eq. (4.2) based on σ_0 and S_0

$$\sigma_1 = \sigma_0 - \tau(S_0) \frac{4l}{d_s} \quad (4.1)$$

$$S_1 = S_0 - f_{CF}^{-1}(\sigma_1)l \quad (4.2)$$

Where, l is the length of segment, d_s is the diameter of CFRP bar, $\tau ()$ is the bond stress-slip function, $f_{CF}^{-1}()$ is the inverse function of CFRP bar stress-strain relation.

- 4) Then the stress and slip of the $(k+1)$ -th segment, σ_k and S_k can also be obtained as follow:

$$\sigma_{k+1} = \sigma_k - \tau(S_k) \frac{4l}{d_s} \quad (4.3)$$

$$S_{k+1} = S_k - f_{CF}^{-1}(\sigma_{k+1})l \quad (4.4)$$

- 5) Because both ends of CFRP bar are well anchored using the equipment shown in Fig. 2-2, the boundary condition is $S_{n+1}=0$. If the calculated $S_{n+1}=0$, the assumed σ_0 is the correct result of σ_C corresponding to the given S_C ; if not, assume a new σ_0 and repeat calculations of step 3) and step 4).

The stress of CFRP bar in base side σ_B can be derived from S_B by the same calculating procedures as component side.

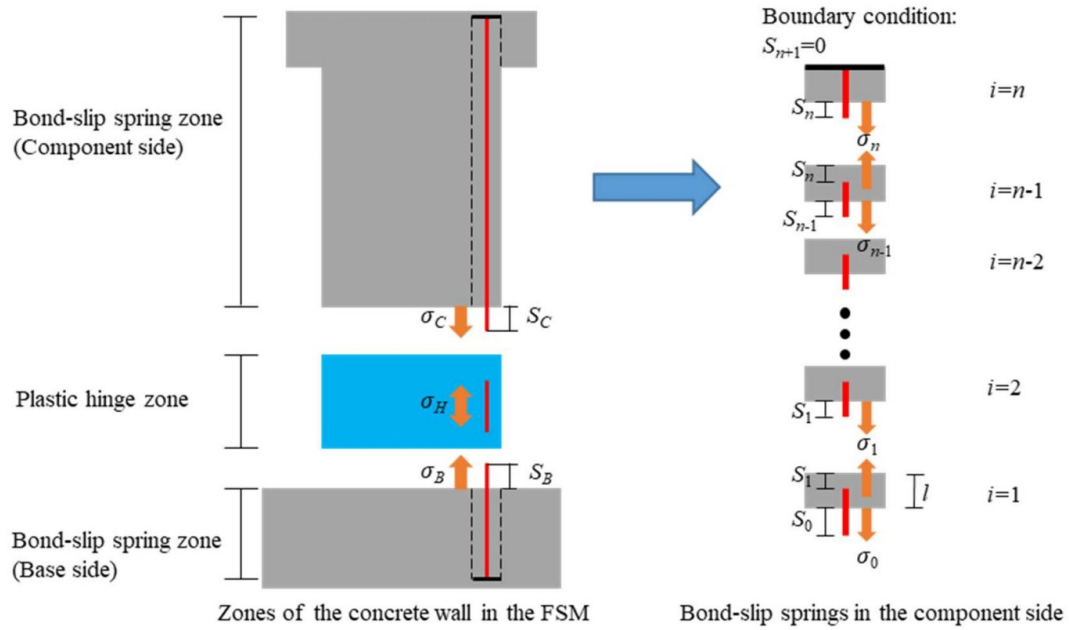


Fig. 4-2 Bond-slip springs model

4.2.3 Plastic hinge zone

In the plastic hinge zone, the stress and strain of CFRP bars are uniformly distributed and the stress in hinge zone equals the stress in the first segment of bond-slip spring zone. In other word, $\sigma_H = \sigma_C = \sigma_B$. CFRP bars are not considered to have any connection with concrete in the plastic hinge zone. While the vertical web reinforcements consisted of deformed steel bars are assumed well bonded with concrete so slippage is ignored. The cross section is divided into finite elements as shown in Fig. 4-3, where the strain profile is also exhibited. It can be seen that only the plane of concrete and steel bars remains plane after bending, the strain profile of CFRP bars is lower than that of concrete and steel bars due to the slip.

The curvature of cross section ϕ can be obtained using the simplified model described in 4.2.1. Once strain at the center of section ϵ_{ce} is decided, the strain of each concrete fiber and steel bar can be calculated based on the plane-remain-plane assumption. Strains of CFRP bars will be obtained using the FSM in 4.2.2. Through obtained strains of reinforcement and concrete, stress will also be got based on material models that will be described in the next section and then the axial load and moment of the section can be calculated.

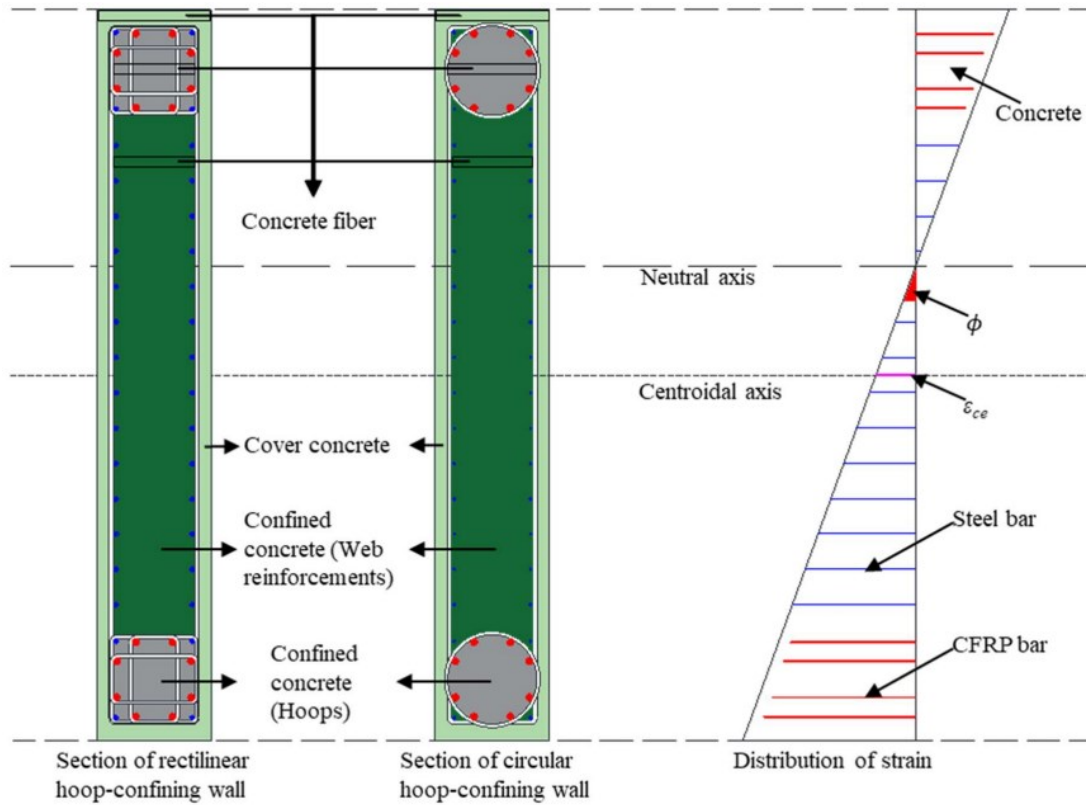


Fig. 4-3 Section discretion and strain profile in the wall section

In the original analysis of SBPDN bars reinforcing concrete walls by Fujitani and Sun [4.3], confinement of hoops in boundary elements was ignored and only considered the confining effect of horizontal web reinforcement. While confinement of hoops placed in boundary elements will be well considered in this dissertation and verified by comparing the analytical and experimental results to confirm that if this modification on the original method [4.3] can enhance the accuracy of analytical method after.

4.2.4 Basic assumptions

Analyzing models in the numerical method have been introduced in previous sections where some assumptions are adopted to simplify calculating procedures. Above-mentioned assumptions will be summarized as follows to clarify preconditions for beginning of the analysis:

- 1) The rotation resulting from flexural deformation concentrates in the plastic hinge zone with uniform curvature.

- 2) The plane of concrete and steel bars remains plane after bending.
- 3) Strain and stress of CFRP bars are uniformly distributed within the plastic hinge zone.

4.2.5 Analytical procedures

Based on above-mentioned assumptions and models, the procedures to trace cyclic V-R response of concrete component considering the bond-slip effect of rebars proposed by Sun et al. [4.1-4.3] can be described as follows:

- 1) Divide a member into bond-slip spring zones and plastic hinge zone as shown in Fig. 4-2.
- 2) For a given drift caused by flexural deformation R_f , calculate curvature ϕ in the plastic hinge zone by Eq. (4.5) based the first assumption and the model shown in Fig. 4-1.

$$\phi = \frac{R_f}{L_p(1 - \frac{L_p}{2L})} \quad (4.5)$$

Where, L_p is the plastic hinge length, L is the shear span.

- 3) Give an initial value to the strain at the center of section ε_{ce} , and obtain the concrete strain distribution ε_c and steel bar strain ε_s across the section depth following the second assumption and Fig. 4-3.
- 4) Give an initial value to the slip S_B at the bond-spring zone of base side.
- 5) Obtain the CFRP stress σ_B at the bond-spring zone of base side by the FSM described in 4.2.2.
- 6) Calculate the CFRP strain ε_{CF} in plastic hinge zone by the equation as below:

$$\varepsilon_{CF} = f_{CF}^{-1}(\sigma_H) \quad (4.6)$$

Where, $f_{CF}^{-1}(\)$ is the inverse function of CFRP bar stress-strain relation, σ_H is the CFRP

bar stress in plastic hinge that equals σ_B and is uniform within the hinge region because of the third assumption.

- 7) Get the slip S_C at the bond-spring zone of component side according to the difference on elongation between CFRP bar and concrete within hinge zone as the below equation shown:

$$S_C = -S_B + (\varepsilon_c - \varepsilon_{CF})L_p \quad (4.7)$$

- 8) Obtain the CFRP bar stress σ_C at the bond-spring zone of component side by the FSM described in 4.2.2.
- 9) If $\sigma_C = \sigma_B$ exists, then the ε_{CF} in step 6) is the CFRP bar strain in hinge zone, if not, assume a new slip S_B and repeat calculations of step 4) through step 8).
- 10) For all CFRP bars, repeat step 4) through step 9) to obtain the CFRP bar strain distribution taking account of the slip effect.
- 11) Calculate the concrete fiber stress f_c , the steel bar stress f_s , and the CFRP bar stress f_{CF} from respective constitutive law that will be introduced in the next section.
- 12) Calculate the internal axial load N by Eq. (4.8)

$$N = \sum f_c A_c + \sum f_s A_s + \sum f_{CF} A_{CF} \quad (4.8)$$

Where, f_c, f_s, f_{CF} , are the stress of each concrete fiber, steel bar, and CFRP bar, respectively, and A_c, A_s, A_{CF} , are the corresponding areas.

- 13) If the calculated internal axial load N balances the applied axial load P within a tolerable error, move forward the next step. If not, assume a new center strain ε_{ce} and repeat from step 3).
- 14) Obtain the moment of cross section using Eq. (4.9).

$$M = \sum f_c A_c h_c + \sum f_s A_s h_s + \sum f_{CF} A_{CF} h_{CF} \quad (4.9)$$

Where, h_c , h_s , h_{CF} are the distances to centroidal axis of each concrete fiber, steel bar, and CFRP bar, respectively.

15) Calculate the lateral force V corresponding to the given drift R_f through Eq. (4.10).

$$V = \frac{M}{L} - PR_f \quad (4.10)$$

Where, P is the applied axial load.

4.2.6 Calculation of flexural deformation and plastic hinge length

As described above, it is inevitable to decide the drift caused by flexural deformation and plastic hinge length before the analysis of hysteretic behavior. In 2.3.3 and 3.3.3 the flexural and shear deformation of specimens have been calculated based on the elastic theory, where the ratio of flexural deformation to overall deformation is 0.83 for all specimens. In this section, flexural deformation and plastic hinge length will be calculated according to the measured vertical displacement along wall height.

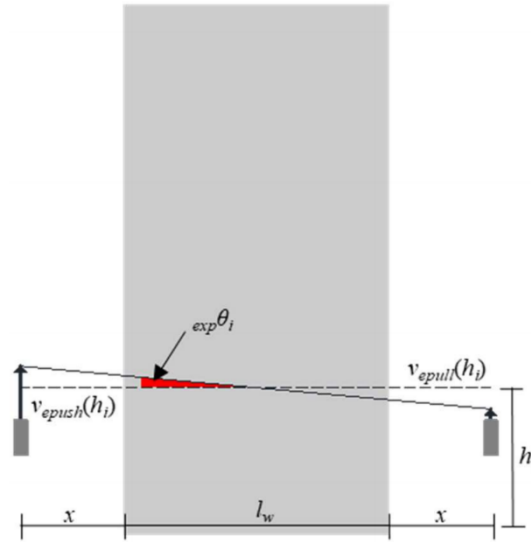


Fig. 4-4 Calculating methods of experimental rotation

The calculating procedures are shown as follows:

- 1) Obtain the experimental rotation $exp\theta_i$ (shown in Fig. 4-4) at the height of 400mm,

800mm and 1200mm according to vertical displacements measured by DTs (shown in Fig. 2-6) at peak drifts by Eq. (4.11)

$$_{exp}\theta_i = \frac{v_{epush}(h_i) + v_{epull}(h_i)}{l_w + 2x} \quad (4.11)$$

Where, $v_{epush}(h_i)$ and $v_{epull}(h_i)$ are the measured displacement by vertical DTs at the height of h_i , l_w is the cross section length of wall, x is the distance between DT and wall.

- 2) Take parameters of α and β that are the ratio of lateral displacement resulting from shear deformation Δ_s to overall lateral displacement Δ and the ratio of plastic hinge length L_p to the section length l_w respectively to obtain the calculating shear rotation $_{cals}\theta_i$ and the calculating flexural rotation $_{calf}\theta_i$ at the height of h_i based on the simplified deformation models shown in Fig. 4-5 by Eq. (4.12) through (4.22)

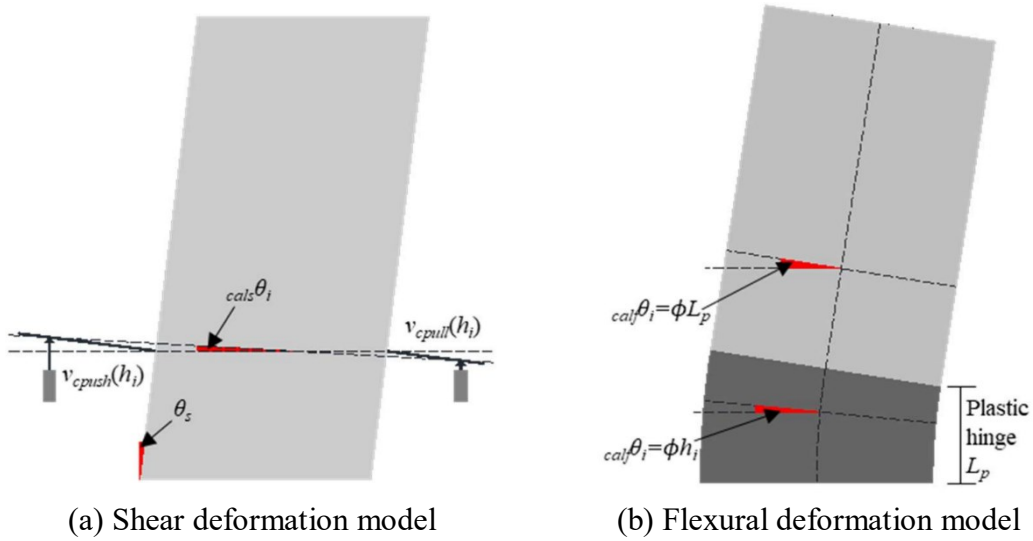


Fig. 4-5 Models of wall deformation

$$\Delta_s = \alpha \Delta \quad (4.12)$$

$$L_p = \beta l_w \quad (4.13)$$

$$\tan \theta_s = \frac{\Delta_s}{L} \quad (4.14)$$

$$\delta(h_i) = \Delta_s \frac{h_i}{L} \quad (4.15)$$

$$v_{cpush}(h_i) = [x + \delta(h_i)] \tan \theta_s \quad (4.16)$$

$$v_{cpull}(h_i) = [x - \delta(h_i)] \tan \theta_s \quad (4.17)$$

$$_{cals}\theta_i = \frac{v_{cpush}(h_i) + v_{cpull}(h_i)}{l_w + 2x} \quad (4.18)$$

$$\Delta_f = \Delta - \Delta_s \quad (4.19)$$

$$\phi = \frac{\Delta_f}{(L - \frac{L_p}{2})L_p} \quad (4.20)$$

$$_{calf}\theta_i = \begin{cases} \phi h_i & h_i \leq L_p \\ \phi L_p & h_i > L_p \end{cases} \quad (4.21)$$

$$_{cal}\theta_i = _{cals}\theta_i + _{calf}\theta_i \quad (4.22)$$

Where, θ_s is the rotation of wall caused by shear deformation, $\delta(h_i)$ is the lateral deformation at the height of h_i resulting from shear deformation, $v_{cpush}(h_i)$ and $v_{cpull}(h_i)$ are the calculated displacement of DTs at the height of h_i , Δ_f is the lateral displacement due to flexural deformation, ϕ is the curvature within plastic hinge region. $_{cal}\theta_i$ is the calculated rotation at the height of h_i .

- 3) $_{cal}\theta_i$ is the function of parameters of α and β and can be presented as the form of $_{cal}\theta_i(\alpha, \beta)$. Calculate quadratic sum ς of the difference value between $_{exp}\theta_i$ and $_{cal}\theta_i$ at each height using the Eq. (4.23).

$$\varsigma = \sum [_{exp}\theta_i - _{cal}\theta_i(\alpha, \beta)]^2 \quad (4.23)$$

- 4) Calculate ς by changing the value of α and β from 0.001 through 0.999 to find the minimum value of ς , to which the corresponding α and β are the calculated results.

The ratio of lateral drift resulting from lateral deformation to overall drift R_f/R versus drift relations for specimens except the specimen CFRPHW3 because of breakdowns of vertical DTs are shown in Fig. 4-6 in red lines by the calculating procedures described above, where the blue lines represent the ratio of flexural lateral drift calculated from elastic theory described in 2.3.3 and 3.3.3. The value of α at different drifts for specimens is shown in Table 4-1, where the average value of all available data is 0.210. The ratio of shear lateral deformation to overall deformation from elastic theory is 0.168. Based on

results obtained from two different methods, the decided value of shear lateral deformation ratio is 0.2 in following analysis, which means that the drift caused by flexural deformation R_f equals $0.8R$, where R is the overall drift.

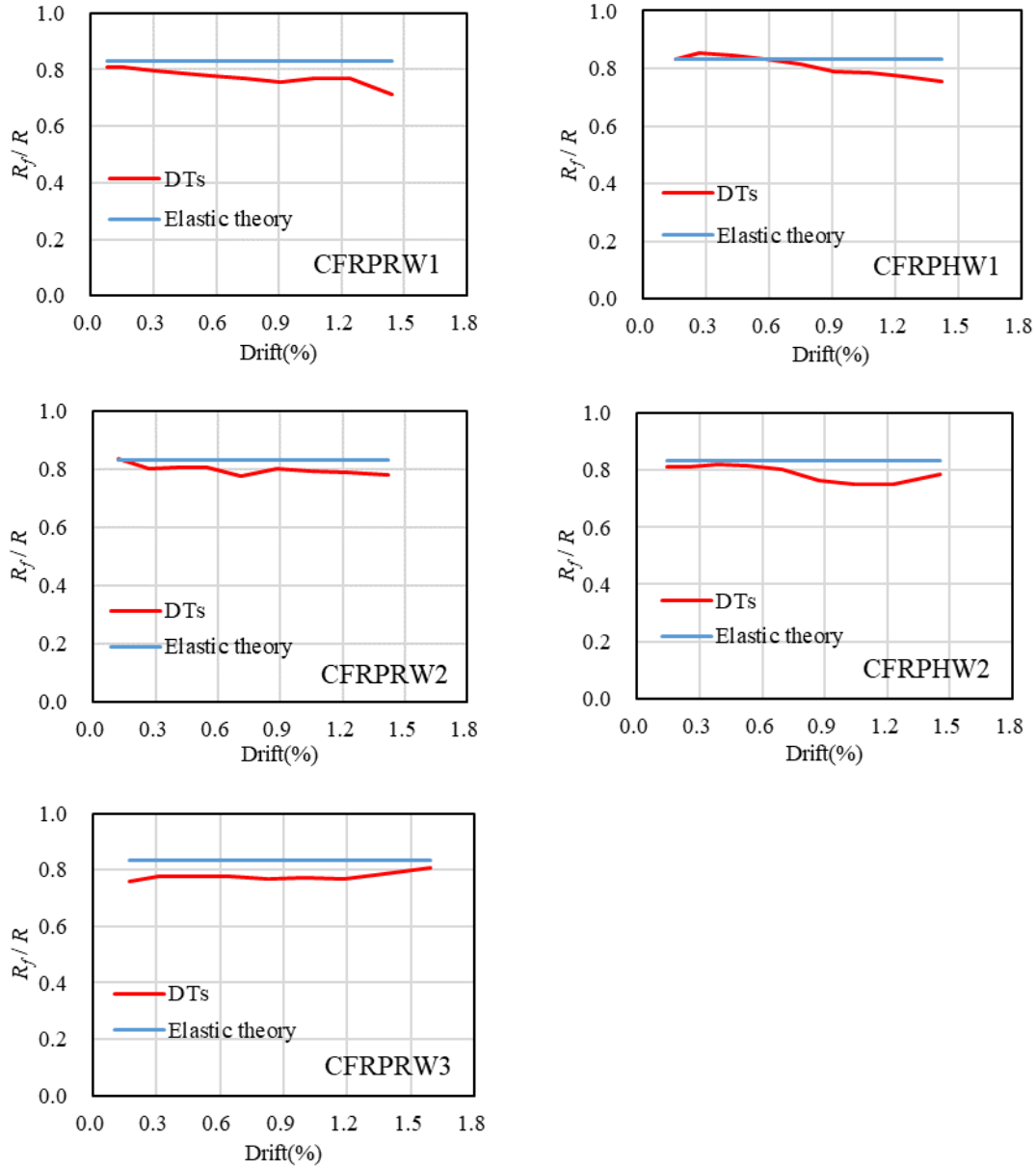


Fig. 4-6 Ratio of flexural lateral deformation to overall deformation

For the ratio of plastic hinge length to section length $\beta = L_p/l_w$, of which relations with drift are shown in Fig. 4-7 in red lines. Specific value of β is also indicated in Table 4-2. The average value of β is 0.585 that is similar with the value of 0.5 adopted by many

researchers according to the review by Bohl [4.4]. So 0.5 is adopted as the hinge length ratio to section length to conduct the analysis on cyclic behavior of concrete walls after.

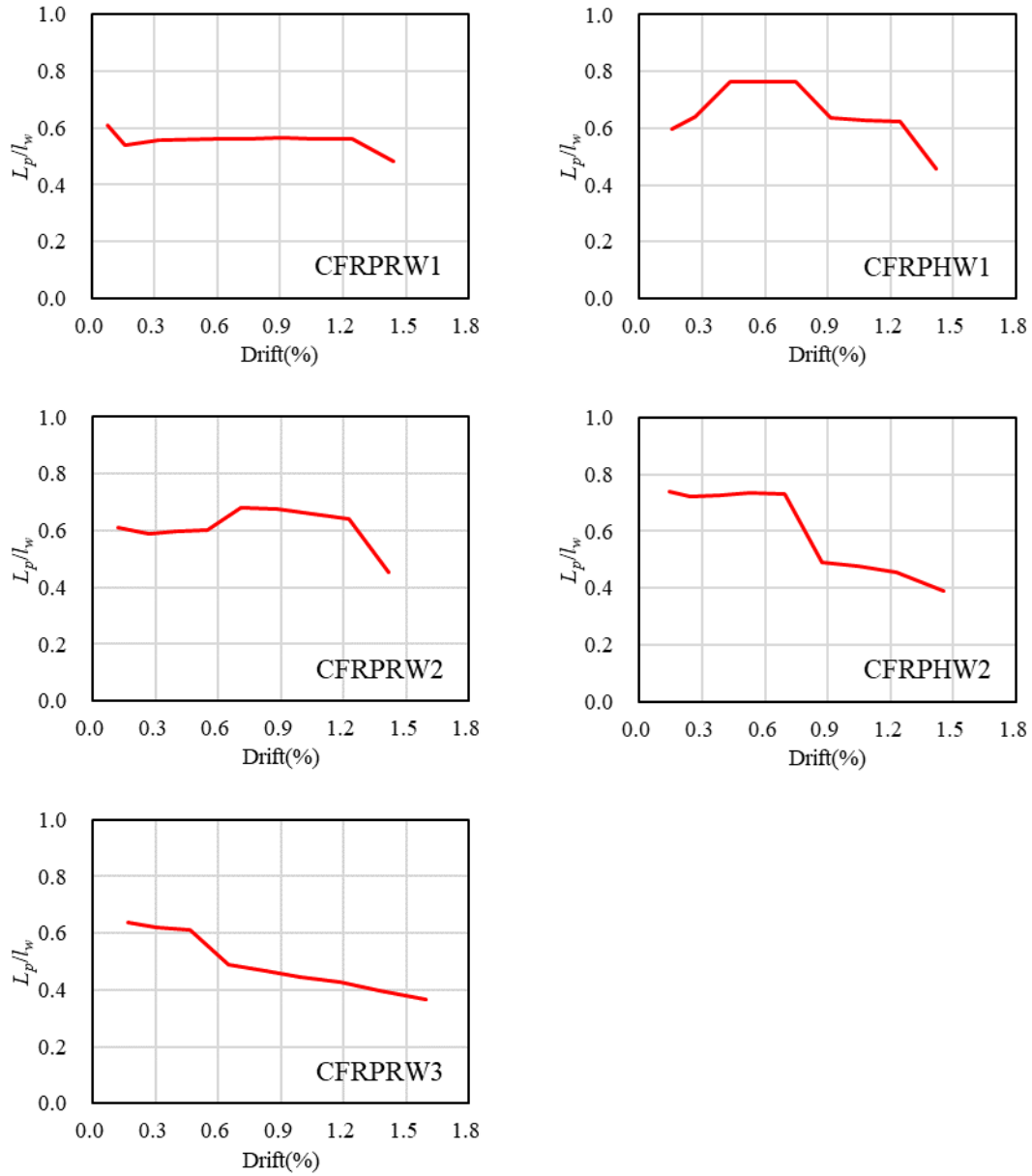


Fig. 4-7 Ratio of plastic hinge length to section length

Table 4-1 Calculated results of α

CFRPRW1	Drift(%)	0.08	0.16	0.32	0.58	0.74
	α	0.191	0.192	0.203	0.225	0.233
	Drift(%)	0.91	1.07	1.24	1.44	Ave.
	α	0.245	0.229	0.232	0.290	0.226

Table 4-1 Continued

CFRPHW1	Drift(%)	0.16	0.27	0.43	0.59	0.75
	α	0.169	0.147	0.156	0.169	0.185
	Drift(%)	0.91	1.08	1.25	1.42	Ave.
	α	0.210	0.217	0.229	0.244	0.191
CFRPRW2	Drift(%)	0.12	0.27	0.40	0.55	0.71
	α	0.163	0.199	0.195	0.195	0.225
	Drift(%)	0.88	1.05	1.22	1.42	Ave.
	α	0.196	0.206	0.212	0.217	0.201
CFRPHW2	Drift(%)	0.14	0.25	0.38	0.53	0.70
	α	0.187	0.189	0.181	0.184	0.196
	Drift(%)	0.87	1.05	1.23	1.45	Ave.
	α	0.237	0.249	0.250	0.214	0.209
CFRPRW3	Drift(%)	0.17	0.31	0.46	0.65	0.83
	α	0.239	0.224	0.223	0.224	0.232
	Drift(%)	1.00	1.18	1.37	1.60	Ave.
	α	0.229	0.232	0.216	0.191	0.223
Ave.	α	0.210				

Table 4-2 Calculated results of β

CFRPRW1	Drift(%)	0.08	0.16	0.32	0.58	0.74
	β	0.611	0.540	0.559	0.560	0.564
	Drift(%)	0.91	1.07	1.24	1.44	Ave.
	β	0.565	0.562	0.561	0.482	0.556
CFRPHW1	Drift(%)	0.16	0.27	0.43	0.59	0.75
	β	0.597	0.642	0.764	0.766	0.763
	Drift(%)	0.91	1.08	1.25	1.42	Ave.
	β	0.637	0.630	0.624	0.457	0.653
CFRPRW2	Drift(%)	0.12	0.27	0.40	0.55	0.71
	β	0.613	0.590	0.598	0.603	0.680
	Drift(%)	0.88	1.05	1.22	1.42	Ave.
	β	0.678	0.658	0.640	0.451	0.612
CFRPHW2	Drift(%)	0.14	0.25	0.38	0.53	0.70
	β	0.742	0.724	0.728	0.736	0.731
	Drift(%)	0.87	1.05	1.23	1.45	Ave.
	β	0.492	0.476	0.455	0.392	0.608
CFRPRW3	Drift(%)	0.17	0.31	0.46	0.65	0.83
	β	0.639	0.622	0.614	0.488	0.469
	Drift(%)	1.00	1.18	1.37	1.60	Ave.
	β	0.447	0.428	0.395	0.367	0.496
Ave.	β	0.585				

4.3 Material models

To complete the analysis of hysteretic behavior of concrete walls reinforced by CFRP bars, constitutive laws of materials consisting of the wall are necessary as described before. Besides stress-strain models of concrete and reinforcement, the bond-slip model of CFRP bar in concrete will also be indicated in this section.

4.3.1 Stress-strain model for confined concrete

As reviewed in the first chapter, many researchers have proposed the stress-strain relationship of confined concrete in the last decades, among which the uniaxial compressive stress-strain envelop curve proposed by Sakino and Sun [4.5] is adopted in this dissertation.

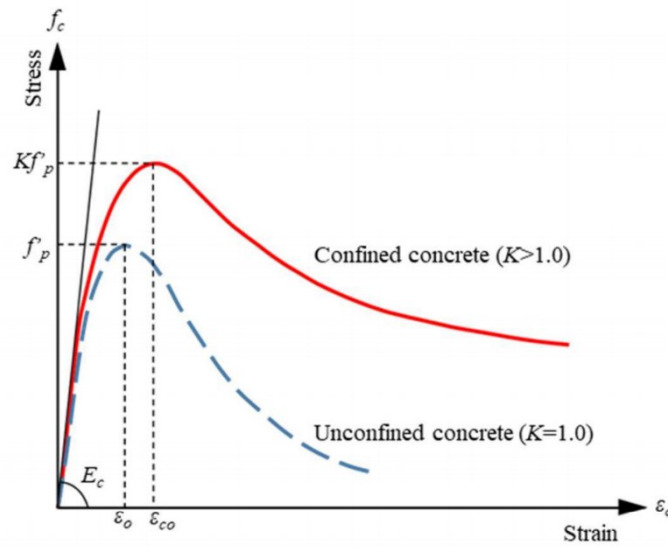


Fig. 4-8 Uniaxial compressive stress-strain envelop curve

Fig. 4-8 indicates the envelop curve of Sakino and Sun model for confined concrete. This model can also be used in unconfined concrete, for example, cover concrete of RC components by giving the value of 1.0 to strength enhancement ratio of confined concrete. The stress-strain relation of confined concrete can be defined by the following Eq. (4.24).

$$f_c = K f'_p \frac{AX + (D - 1)X^2}{1 + (A - 2)X + DX^2} \quad (4.24)$$

Where, f'_p is the plain concrete strength, K is the strength enhancement ratio that can be calculated by Eq. (4.25) and Eq. (4.26) [4.6], $X=\varepsilon_c/\varepsilon_{co}$, f_c and ε_c are the stress and strain, ε_{co} is the strain at the peak, which can be obtained by Eq. (4.27), $A=E_c/E_{sec}$, E_c is the Young's modulus of elasticity of concrete from Eq. (4.29) and E_{sec} is the secant modulus at the point of peak stress from Eq. (4.30), D is the parameter governing the slope of descending portion of the stress-strain curve, which is decided using Eq. (4.31).

$$K = 1 + 11.5 \frac{\rho_h f_{yh}}{f'_p} \left(\frac{d''}{C} \right) \left(1 - \frac{s}{2D_c} \right) \quad \text{for rectilinear hoop} \quad (4.25)$$

$$K = 1 + 2.05 \frac{\rho_h f_{yh}}{f'_p} \left(1 - \frac{s}{2D_c} \right)^2 \quad \text{for circular hoop} \quad (4.26)$$

Where, ρ_h is the volumetric ratio of hoop to the confined core measured center-to-center of perimeter transverse reinforcement, f_{yh} is the yield strength of transverse reinforcement, d'' and C are the nominal diameter and laterally unsupported length of transverse reinforcement, respectively, s is the spacing of transverse reinforcement, and D_c is the distance between centroids of perimeter transverse reinforcement.

$$\frac{\varepsilon_{co}}{\varepsilon_o} = \begin{cases} 1 + 4.7(K - 1) & K \leq 1.5 \\ 3.35 + 20(K - 1.5) & K > 1.5 \end{cases} \quad (4.27)$$

Where, ε_o is the strain at maximum stress of unconfined concrete that can be obtained by the following equation Eq. (4.28).

$$\varepsilon_o = 0.94(f'_p)^{0.25} \times 10^{-3} \quad (4.28)$$

$$E_c = (0.69 + 0.332 \sqrt{f'_p}) \times 10^4 \quad (4.29)$$

$$E_{sec} = \frac{K f'_p}{\varepsilon_{co}} \quad (4.30)$$

$$D = 1.5 - 0.017 + 1.6 \sqrt{(K - 1)f'_p/23} \quad (4.31)$$

There are also several points needed to be noted. First, the unit of stress in this model is MPa and the tensile stress is always 0, in other words, concrete does not resist tensile

stress. Second, the size effect is ignored here, so the unconfined concrete strength f'_p equals the compressive strength of concrete cylinder f'_c (200mm height by 100 mm diameter) that is obtained by multiplying a conversion coefficient of 0.86 and the measured cubic strength f'_{cu} according to European standard [4.7] and Chinese standard [4.8]. The calculated cylinder strength f'_c is 49.4MPa that will be used in the analysis of this chapter. The last one is that the parameter D in original Sun and Sakino model was modified by Tian, Sakino and Sun to take into account of the effect of strain gradient across the section by Eq. (4.32) [4.9]. The modified D from Eq. (4.32) will be adopted to replace Eq. (4.31) in the analysis after.

$$D = 2.25 - 0.017 + 1.6 \sqrt{(K - 1)f'_p/23} \quad (4.32)$$

To assess cyclic behavior of concrete walls, Kitajima and Sun also proposed cyclic (unloading and reloading) stress-strain rules for confined concrete [4.10]. Fig. 4-9 shows the unloading and reloading paths. As unloading occurs at the point A (ε_{un}, f_{un}), the unloading curve is assumed to be a parabola with the peak point B ($\varepsilon_{pl}, 0$) where the tangential stiffness is zero. The reloading curve is assumed to be a straight line connecting the reloading point R (ε_{un}, f_{un}) with the point C ($\varepsilon_{un}, 0.9f_{un}$) and along that straight line return to the point D on the envelop curve. Based on these assumptions, the unloading curve (AB) and reloading curve (RD) can be decided by Eq. (4.33)

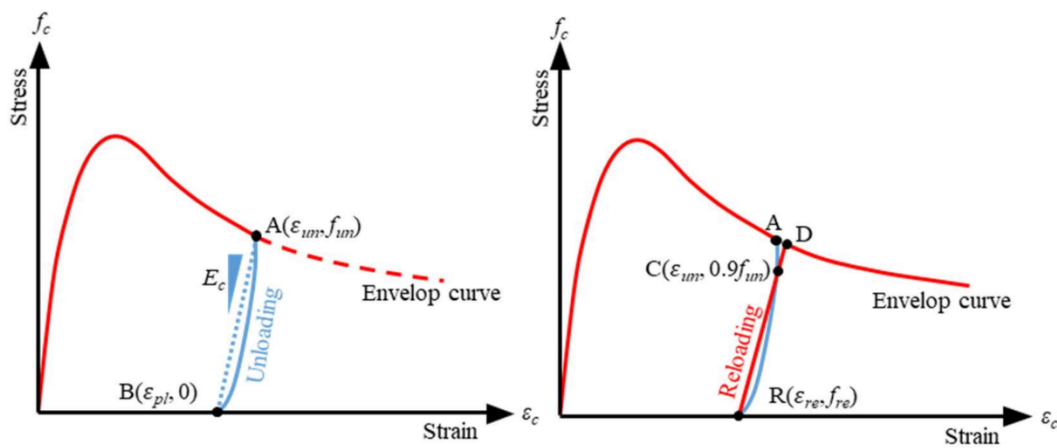


Fig. 4-9 Complete stress-strain curve of confined concrete under compression

$$f_c = \begin{cases} f_{un} \left(\frac{\varepsilon_c - \varepsilon_{pl}}{\varepsilon_{un} - \varepsilon_{pl}} \right)^2 & \text{unloading} \\ f_{re} + \left(\frac{0.9f_{un} - f_{re}}{\varepsilon_{un} - \varepsilon_{re}} \right) (\varepsilon_c - \varepsilon_{re}) & \text{reloading} \end{cases} \quad (4.33)$$

Where, ε_{pl} is the plastic residual strain after unloading, which is defined as the abscissa at the intersection of a straight line with the slope of E_c from the point A with the strain axis and can be calculated by Eq. (4.34).

$$\varepsilon_{pl} = \varepsilon_{un} - \frac{f_{un}}{E_c} \quad (4.34)$$

4.3.2 Stress-strain model for reinforcement

CFRP bars have a completely linear-elastic stress-strain relation. It is simple to obtain the CFRP bar stress through multiplying the elastic modulus and the stain. For deformed steel bars consisting of web reinforcement, the bilinear model of perfect elastoplasticity without considering strain-hardening effect is adopted to simplify calculating procedures.

4.3.3 Bond-slip model of CFRP bar in concrete

The bond-slip relation is significant in the numerical analysis considering the effect of slippage. Funato and sun have modeled the bond stress-slippage relation of SBPDN rebars, which is an WBUS steel bars with the yield strength of 1450 MPa and bond strength of 3MPa, based on experimental results [4.2]. That bond-slip model of SBPDN rebars proposed by Funato and Sun will be adopted and modified in the analysis of CFRP bars reinforcing walls.

Fig. 4-10 shows the complete bond stress-slip relationship of SBPDN rebar. As shown in Fig. 4-10 (a) there are two envelop curves for monotonic loading and cyclic loading, respectively. Envelop curves for both monotonic loading and cyclic loading decline by the slope of $\tau_{max}/1000S_{\tau_{max}}$ until the residual stress of $\tau=\alpha\tau_{max}$ ($\alpha=0.13$) after which the envelop curves become a straight line that is parallel to S axis with the constant residual bound stress. The bond strength τ_{max} and corresponding slip $S_{\tau_{max}}$ are 3.0MPa, and

0.015mm, respectively according to Funato and Sun [4.2]. To apply that model to CFRP bars, τ_{max} and $S_{\tau_{max}}$ are both modified based on the investigation conducted by Quayyum who studied bond behavior of CFRP bars in concrete and proposed calculating equations, Eq. (4.35) and Eq. (4.36), for τ_{max} and $S_{\tau_{max}}$ of CFRP bars [4.11]. The equations proposed by Quayyum consider the effects of type of rebar surface, concrete strength, concrete cover, embedment length and confinement on concrete, which exhibit a well agreement with experimental results obtained from hundreds of specimens. Based the equations from Quayyum, τ_{max} and $S_{\tau_{max}}$ of CFRP bars in bond-slip spring zone of the component side are 4.0 MPa and 4.0 mm, respectively, which differ from that in bond-slip spring zone of the base side with the value of 6.0 MPa and 0.7 mm because of the difference on embedment length of CFRP bars. The embedment length of CFRP bars here means the length of bond-slip spring zone.

Fig. 4-10 (b) shows unloading and reloading curves before the peak point B. As unloading occurs at the point E on the envelop curve, the bond stress decreases linearly by the slope of initial stiffness to the point F ($\tau = -\alpha\tau_{max}$) and then move parallelly to S axis until return to the opposite envelop curve (example of path: $E \rightarrow F \rightarrow G \rightarrow A'$). If reloading occurs during that unloading path, for example, from the point H then the reloading curve go along the initial stiffness and come back to the envelop curve (example of path: $E \rightarrow F \rightarrow H \rightarrow I \rightarrow B$).

Unloading and reloading curves after experiencing the peak point B are shown in Fig. 4-10 (c). Unloading from J to K takes the initial stiffness as descending slope, after which unloading curve moves along the parallel line to S axis and then goes to the opposite cyclic loading envelop curve once the slip increases reversely (example of path: $J \rightarrow K \rightarrow L \rightarrow D' \rightarrow M$). The reloading curve goes up by the initial stiffness to the residual stress with which moves horizontally intersecting with τ axis at the point O. With increasing slip, the bond stress rises along a straight line connecting the point O and point Q return to the cyclic loading envelop curve, where the bond stress at point Q can be obtained by Eq. (4.37) from coordinates of the point J (S_J, τ_J) experiencing maximum slip (example of path: $M \rightarrow N \rightarrow O \rightarrow Q \rightarrow P$). While if reloading occurs on the halfway, the reloading curve goes by the initial stiffness up to the intersecting point S with OP line and then progresses linearly to the point P (example of path: $J \rightarrow K \rightarrow R \rightarrow S \rightarrow Q \rightarrow P$).

The final Fig. 4-10 (d) indicates unloading and reloading curves after the envelop curve declined to the residual stress with the large slip. Lines of WY and VU both have the slope of 0 to prevent the slope of reloading curve from becoming negative during the analysis in the situation of large slip.

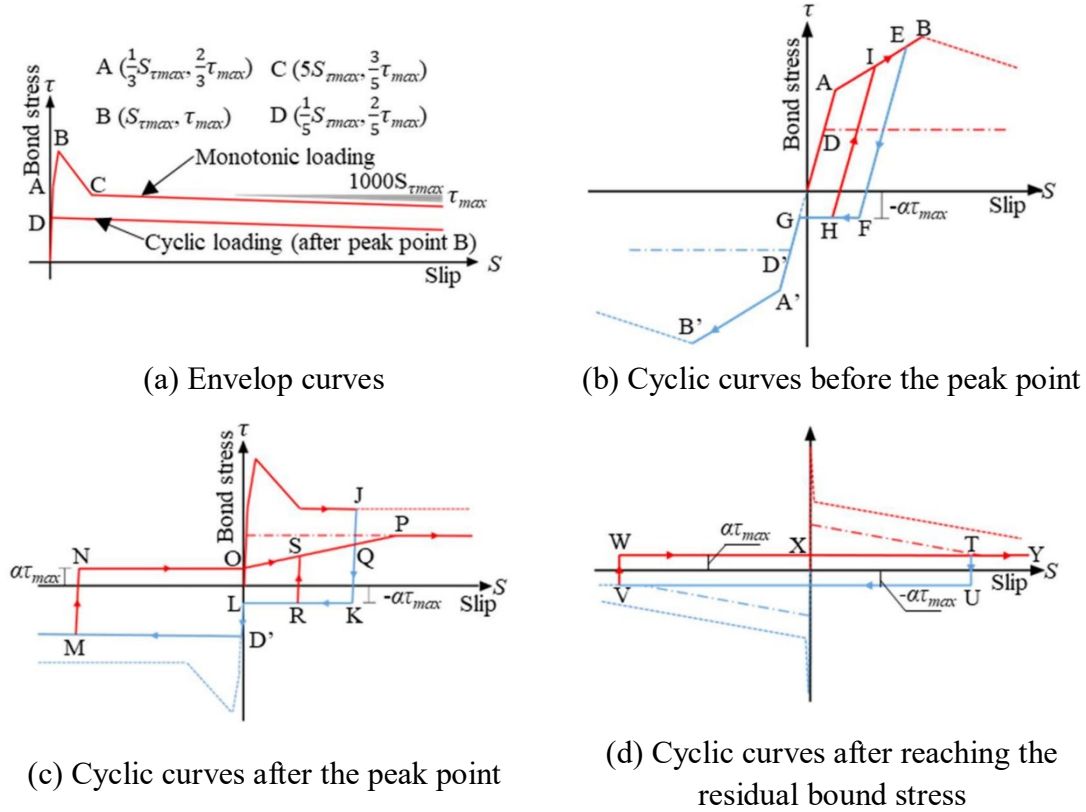


Fig. 4-10 Bond stress-slip model

$$\frac{\tau_{max}}{\sqrt{f'_c}} = 0.03 + 0.14 \frac{c}{d_b} + 9.0 \frac{d_b}{l_{embed}} + 2.9 \frac{A_{tr}}{s n d_b} \quad (4.35)$$

$$\frac{S_{\tau max}}{l_{embed}} = \frac{\eta}{1000} (20.8 - 1.3 \sqrt{f'_c} - 2.1 \frac{c}{d_b} - 3.8 \frac{A_{tr}}{s n d_b}) \quad (4.36)$$

Where, f'_c is the compressive strength of concrete in MPa, c is the concrete cover in mm, d_b is the CFRP bar diameter in mm, l_{embed} is the embedment length of CFRP bar in mm, A_{tr} is the area of transverse reinforcement in mm², s is the spacing of transverse reinforcement in mm, n is the number of CFRP bars being developed along the plane of splitting, η is the rebar surface modification factor (=0.43).

$$\tau_Q = \beta f_{cycle}(S_I) \quad (4.37)$$

Where, $f_{cycle}(S_I)$ is the function of cyclic loading envelop curve, β is the decline ratio of bond stress due to the effect of cyclic loading, here β equals 0.8.

4.4 Verification of the numerical analytical model

The modifications on analytical method proposed originally by Kitajima, Funato, Fujitani and Sun [4.1-4.3] are summarized in Table 4-3. To verify the necessity of considering slip effects of CFRP bars and reasonability of modifications. The analytical V-R curves without considering slip effects will be compared with experimental results first and then comparisons between modification cases and the original method will also be conducted.

Table 4-3 Details of modifications for CFRP bars reinforcing concrete walls

No.	Item	Original [4.1-4.3]	Modification
1	Bond strength of WBUS bar	$\tau_{\max}=3.0\text{MPa}$	$\frac{\tau_{\max}}{\sqrt{f'_c}} = 0.03 + 0.14 \frac{c}{d_b} + 9.0 \frac{d_b}{l_{embed}} + 2.9 \frac{A_{tr}}{s n d_b}$
2	Corresponding slip to bond strength	$s_{\tau_{\max}}=0.015\text{mm}$	$\frac{s_{\tau_{\max}}}{l_{embed}} = \frac{\eta}{1000} \left(20.8 - 1.3\sqrt{f'_c} - 2.1 \frac{c}{d_b} - 3.8 \frac{A_{tr}}{s n d_b} \right)$
3	Confinement from hoops in boundary elements	Ignoring	Taking into consideration

4.4.1 Effect of the bond-slip between CFRP bar and concrete

Fig. 4-11 shows the comparison between analytical V-R curves ignoring slip effect and measured ones. The analytical method without consideration of slip adopts the same analytical method and material models with the analytical method considering slip except the FSM. Strains of CFRP bars are determined based on the plane-remain-plane assumption in the analysis ignoring slip. That analytical method ignoring slip effect is widely used in the analysis on ordinary RC components. It can be seen from Fig. 4-11 that analytical hysteretic curves overestimate measured experimental results in spite of the axial load ratio and form of hoop.

The overestimation on lateral resistance using the analytical method ignoring bond-slip of CFRP bars can be account for overlarge CFRP strains obtained from analysis as shown

in Fig. 4-12. That overestimated CFRP strains can be attributed to the ignorance of slippage of CFRP bars.

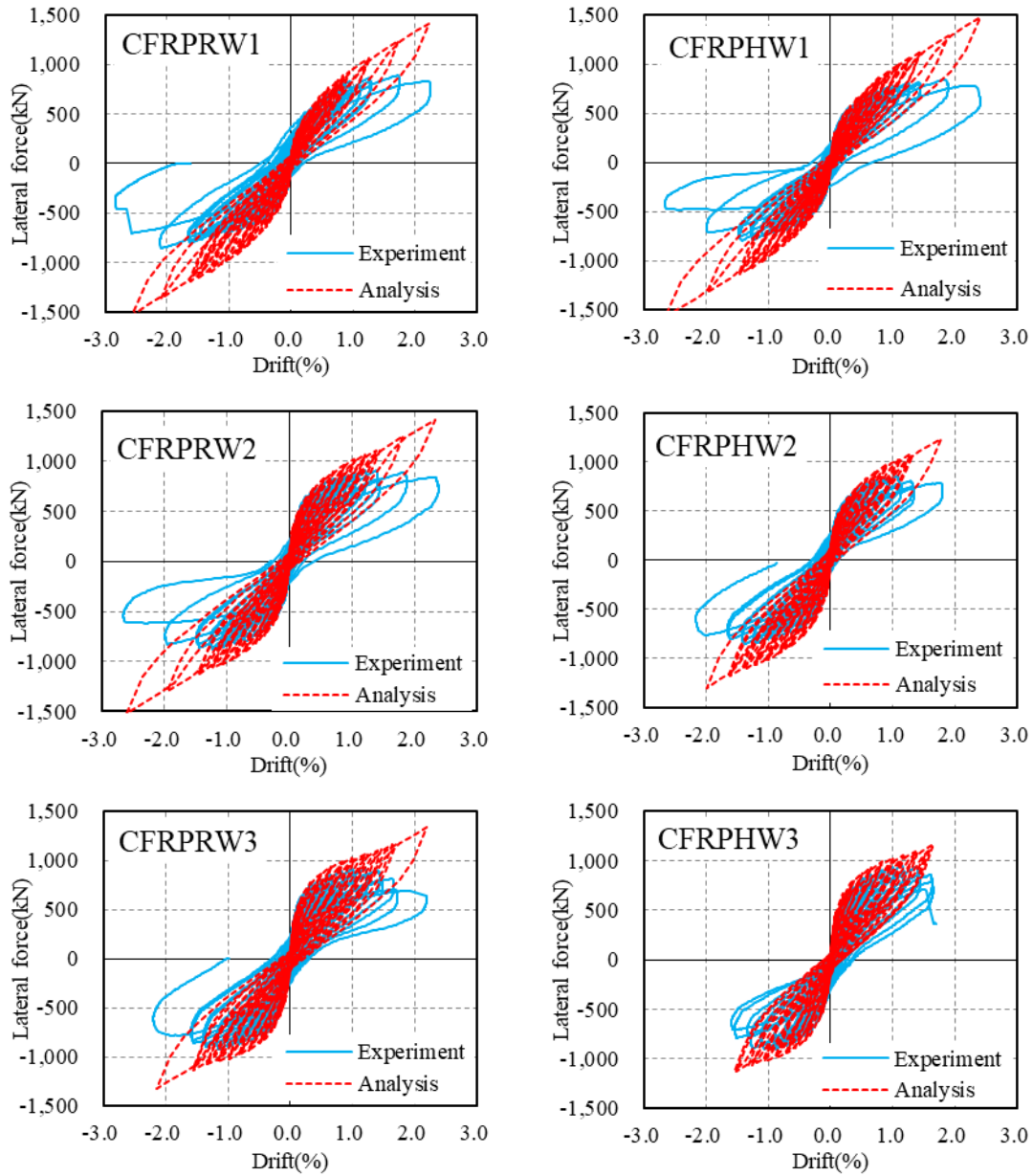


Fig. 4-11 Comparison between experimental and analytical V-R curves ignoring bond-slip of CFRP bars

Based on the comparison between analytical results without consideration of slip effect and experimental results, it is clearly that taking into account of the bond-slip of CFRP bars is necessary in analyzing cyclic behavior of walls reinforced by CFRP bars.

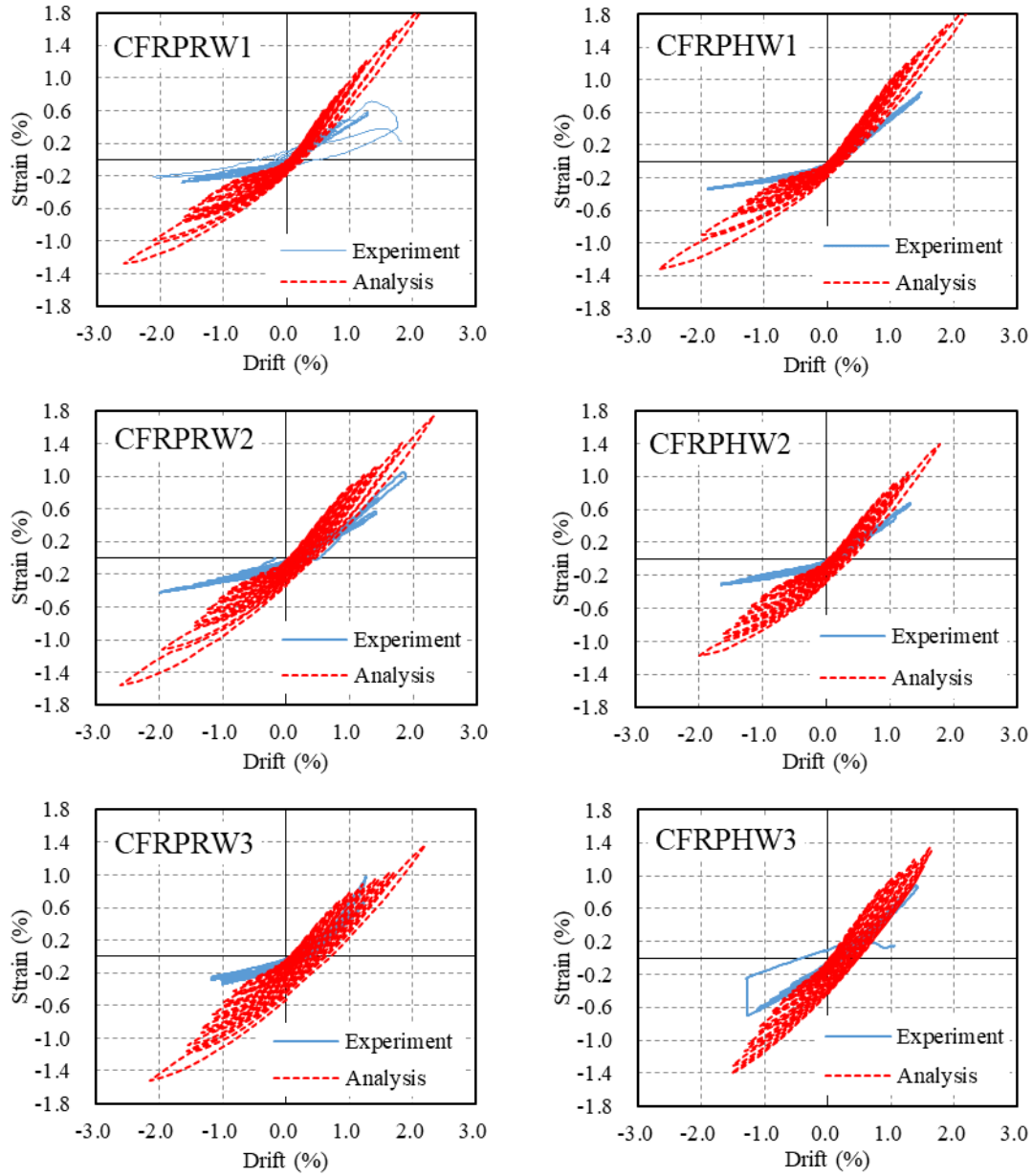


Fig. 4-12 Comparison between experimental and analytical CFRP bar strains ignoring bond-slip of CFRP bars

Fig. 4-13 shows analytical hysteretic curves considering the bond-slip effect with original bond strength and corresponding slippage, besides the results with modified bond strength and corresponding slippage are also indicated to verify validity of modifications on bond-slip characteristics. Analytical cyclic V-R responses with the modified bond stress τ_{max} and corresponding slip $S_{\tau_{max}}$ agree much better with experimental results than that with the original τ_{max} and $S_{\tau_{max}}$ for all specimens.

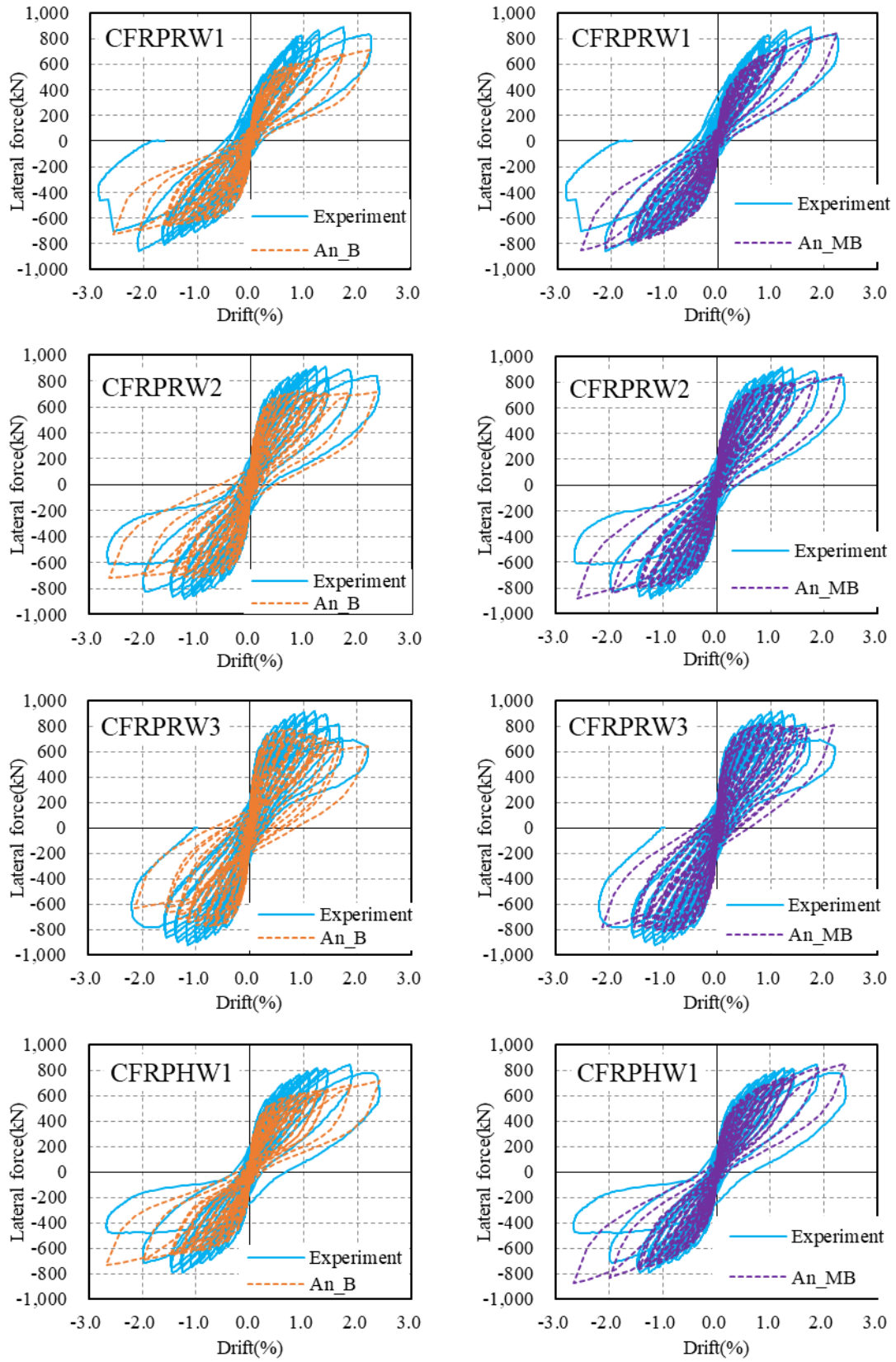


Fig. 4-13 Effect of bond-slip characteristics on cyclic behavior of walls

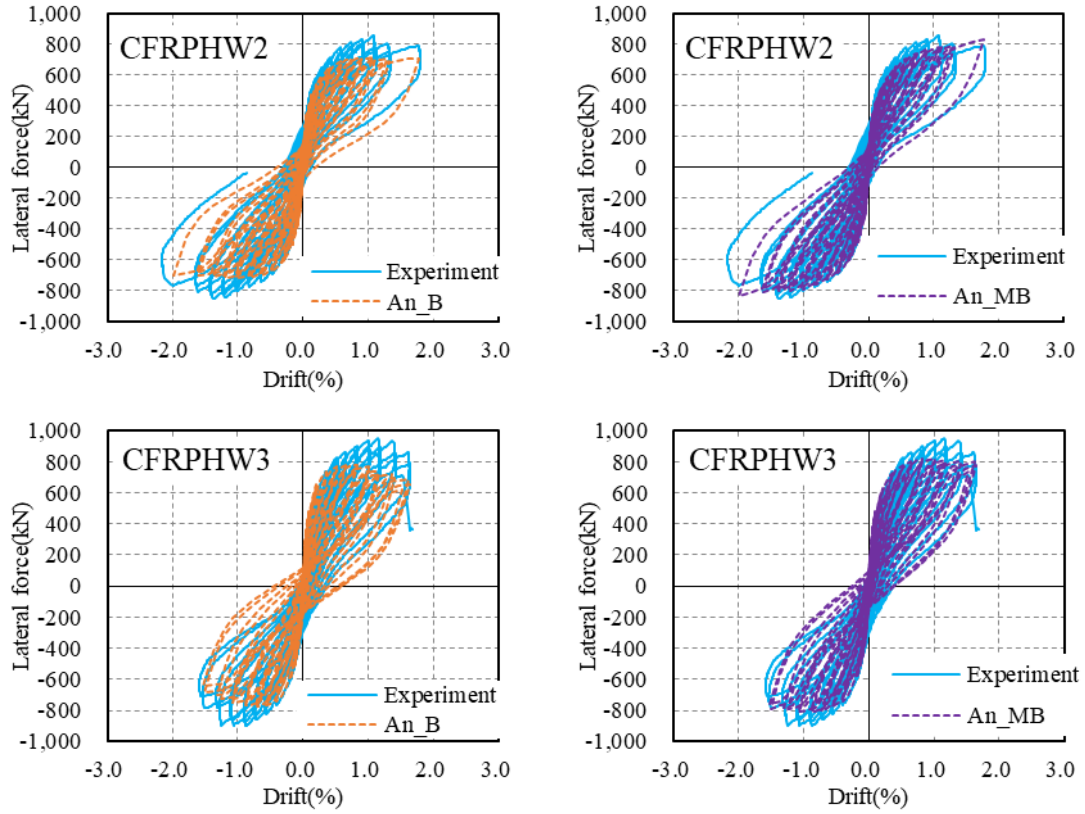


Fig. 4-13 Continued

Where, the legend “Experiment” represents the experimental result, “An_B” means the analytical result with original bond strength (3.0MPa) for SBPDN rebars and corresponding slip for SBPDN rebars, and “An_MB” expresses the analytical result with modified bond strength and corresponding slip for CFRP bars.

According to the analytical CFRP strains obtained from two cases of An_B and An_MB shown in Fig. 4-14, the reason resulting in underestimation for lateral force in the case of An_B is underestimating CFRP strains. It can be inferred that the bond-slip parameters of bond strength τ_{max} and corresponding slippage $S_{\tau_{max}}$ with value of 3.0MPa and 0.015mm for SBPDN rebars proposed by Funato and Sun [4.2] give an underestimation on CFRP bar strains resulting in the same trend on lateral force, while results using the modified τ_{max} (4MPa for the component side, 6MPa for the base side) and $S_{\tau_{max}}$ (4.0mm for the component side, 0.7mm for the base side) based on reference [4.11] agree well with experimental results of both V-R curves and CFRP strains.

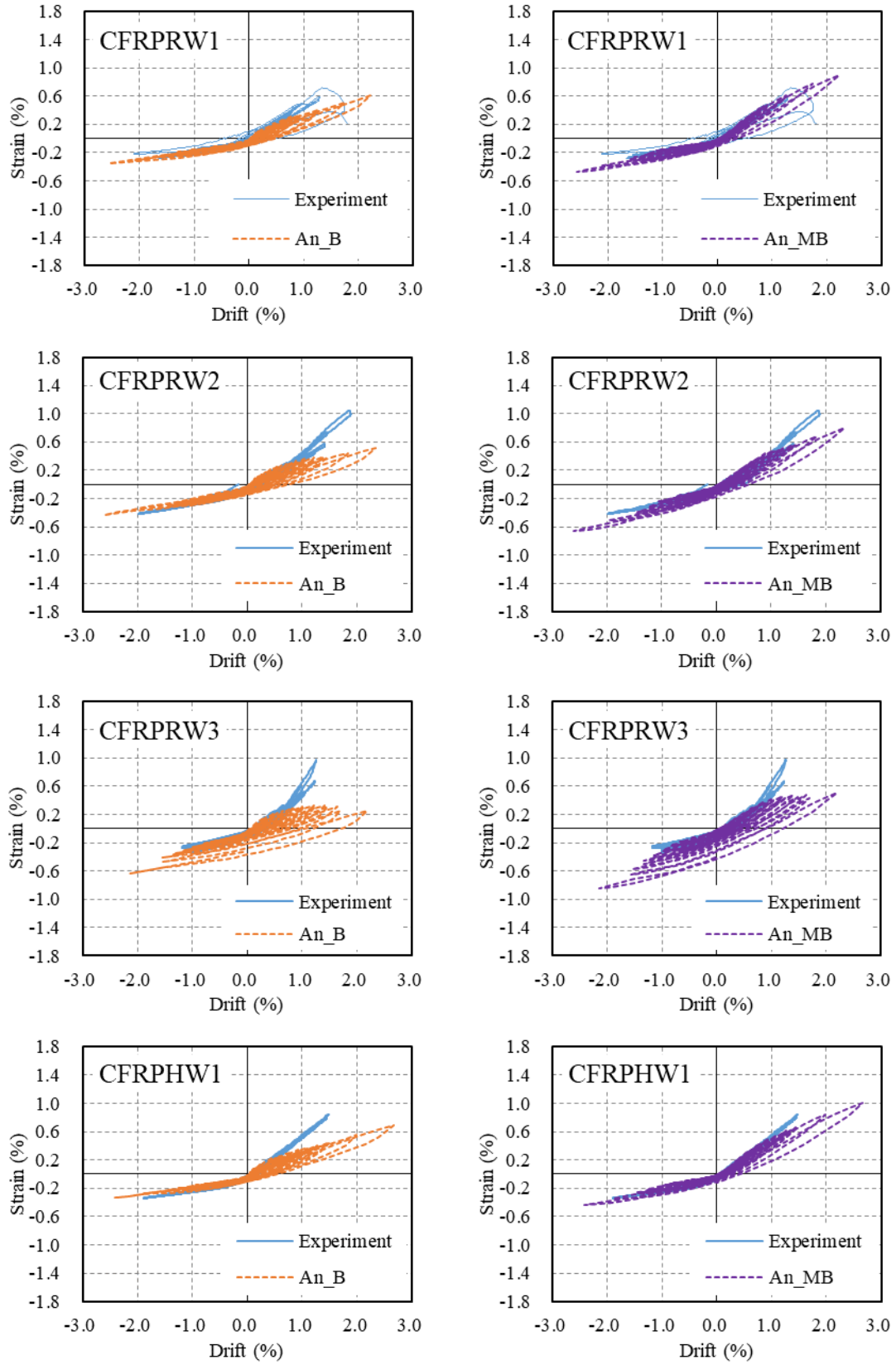


Fig. 4-14 Effect of bond-slip characteristics on the CFRP bar strain

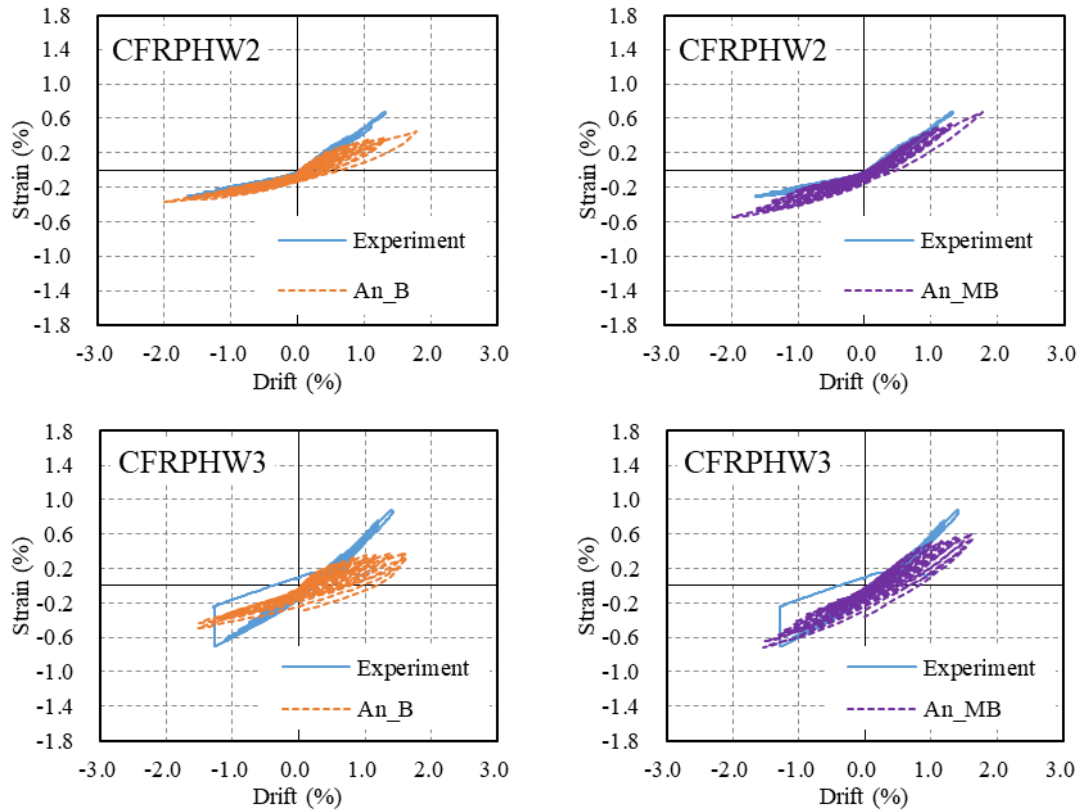


Fig. 4-14 Continued

Where, the legend “Experiment” represents the experimental result, “An_B” means the analytical result with original bond strength (3.0MPa) for SBPDN rebars and corresponding slip for SBPDN rebars, and “An_MB” expresses the analytical result with modified bond strength and corresponding slip for CFRP bars.

4.4.2 Effect of the confinement from hoops in boundary elements

Fig. 4.15 shows comparisons of V-R curves obtained by analytical cases considering confinement from boundary hoops or not, both analytical cases here take the modified bond strength and corresponding slippage. The case considering boundary hoop confinement exhibit relatively high accuracy compared with that without hoop confinement for lateral force-drift responses.

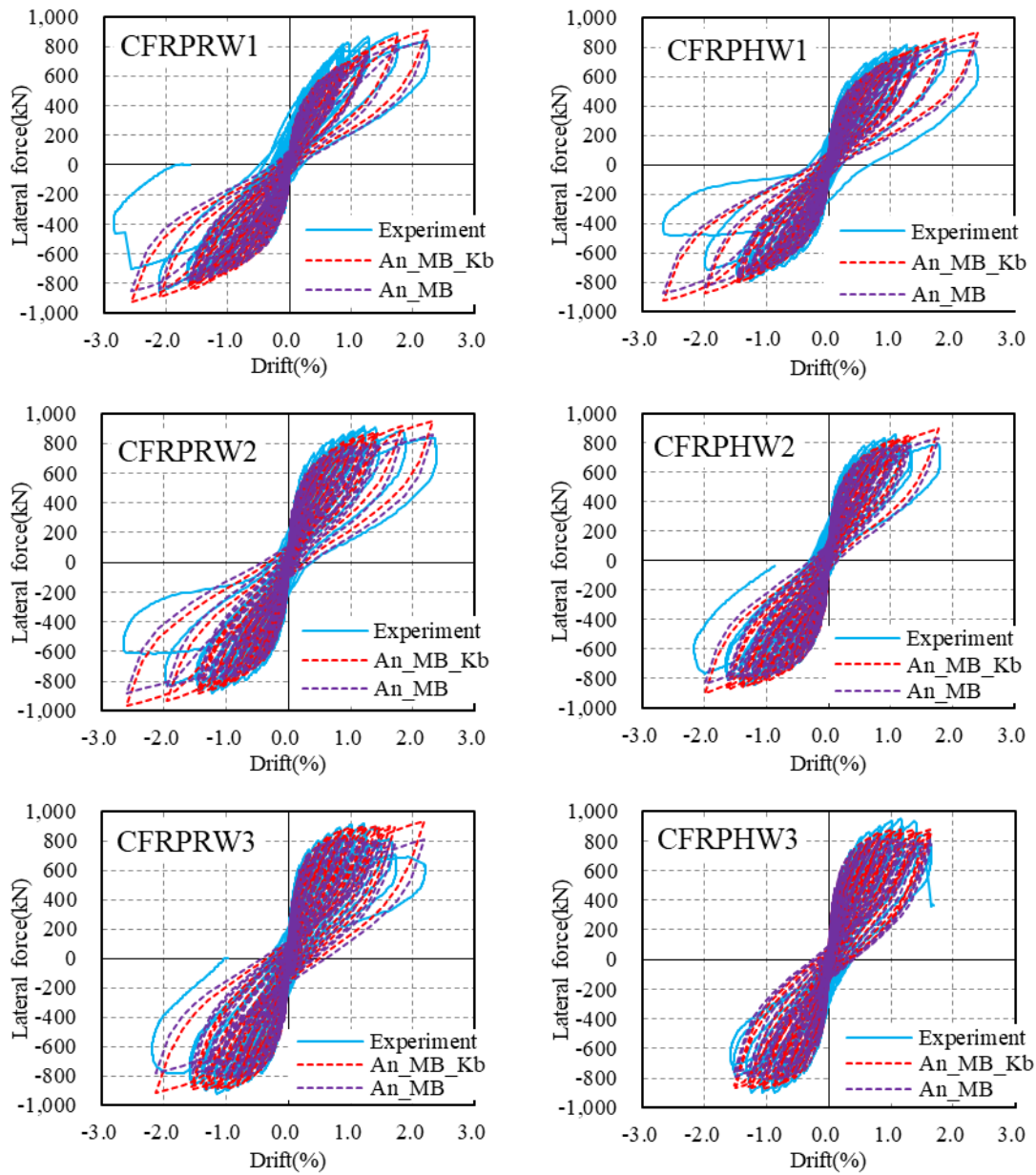


Fig. 4-15 Effect of confinement from boundary element hoops on cyclic behavior

Where, the legend “Experiment” represents the experimental result, “An_MB” is the analytical result with modified bond strength and corresponding slip, “An_MB_Kb” is the analytical result with modified bond strength, to which corresponding slip, and consideration of confinement from boundary hoops.

To better understand the effect of confinement from hoops in boundary element, Fig. 4-16 compares envelop curves of An_MB and An_MB_Kb, where the black circle

superposed represents the last point before compressive failure of CFRP bar commencing. Table 4-4 also shows comparisons of analytical cases with experiments for the lateral force at circling points in Fig. 4-16. The conclusion can be drawn from that comparisons that considering the confinement provided by boundary hoops can improve accuracy of the analytical capacity by 12% at maximum.

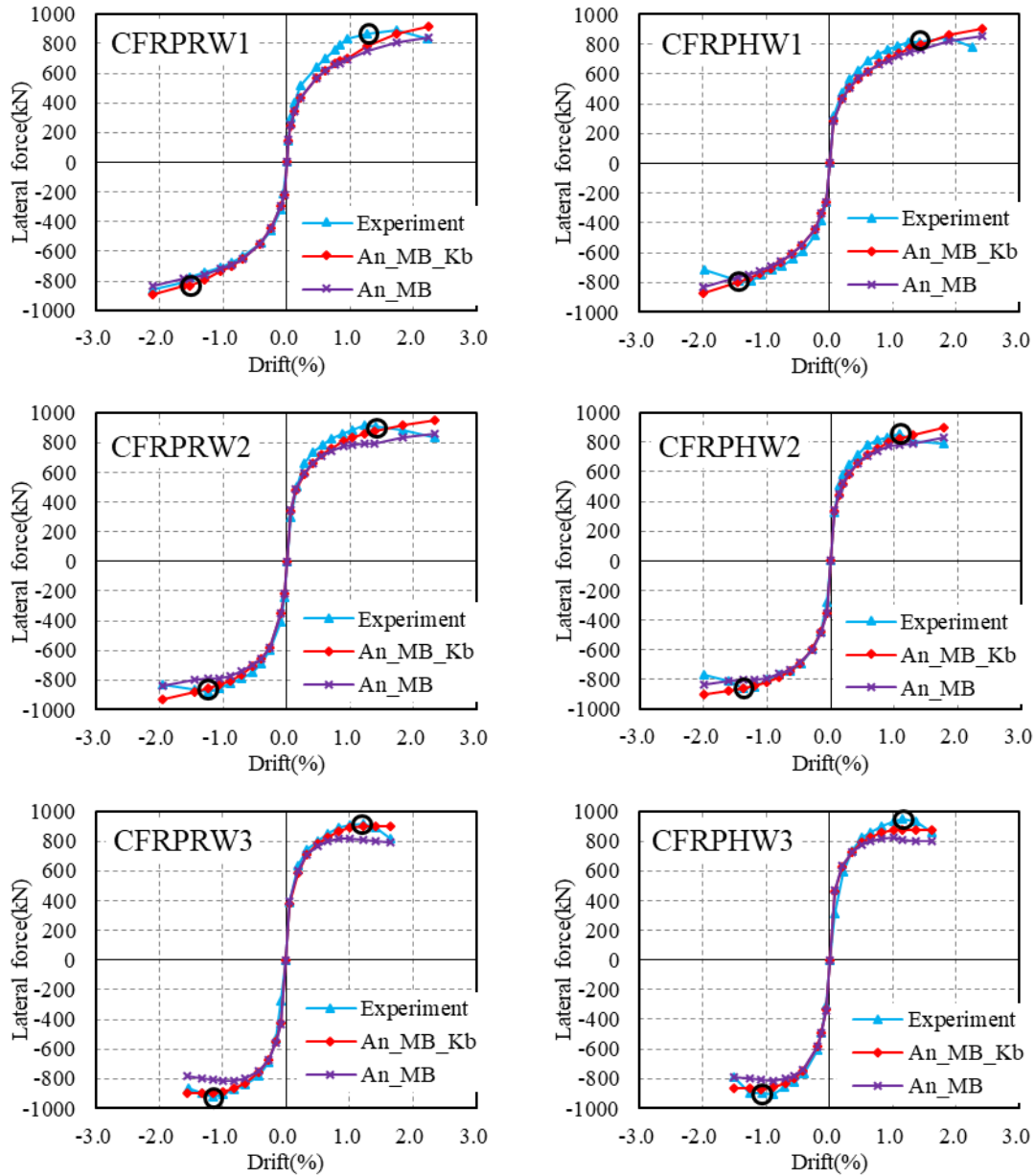


Fig. 4-16 Effect of confinement from boundary element hoops on envelop curves

Where, the legend “Experiment” represents the experimental result, “An_MB” is the

analytical result with modified bond strength and corresponding slip, “An_MB_Kb” is the analytical result with modified bond strength, to which corresponding slip, and consideration of confinement from boundary hoops.

Table 4-4 Analytical lateral forces at the drift before CFRP bar failure

Notation	V_{exp} (kN)		V_{exp}/V_{an}		V_{exp}/V_{anKb}	
	+	-	+	-	+	-
CFRPRW1	866.07	806.50	1.16	1.04	1.11	0.97
CFRPHW1	812.08	790.97	1.06	1.03	1.01	0.99
CFRPRW2	908.92	880.24	1.14	1.11	1.04	1.03
CFRPHW2	859.59	854.98	1.10	1.07	1.04	1.00
CFRPRW3	920.52	923.60	1.13	1.15	1.02	1.03
CFRPHW3	950.91	896.46	1.17	1.11	1.09	1.03

Notes: V_{exp} is the lateral force of the last drift level before compressive failure of CFRP bar, to which V_{anKb} and V_{an} are the corresponding analytical lateral forces considering confinement from boundary hoops or not, respectively, + and – mean the push and pull directions respectively.

Fig. 4-17 shows the effect of boundary hoop confinement on residual drift. In general, no apparent difference on residual drift can be observed between two analytical cases considering confinement provided by boundary hoops or not before the drift of 0.8%, after which, analytical residual drifts without boundary confinement increase more sharply than that considering boundary confinement in spite of the axial load ratio and hoop form. Besides, before 0.8% drift, An_MB_Kb and An_MB underestimate residual drifts for all specimens that trend remains for An_MB_Kb except the specimen of CFRPHW3 as increasing drift, while overestimation by An_MB can be observed for specimens CFRPRW2, CFRPRW3 and CFRPHW3.

Better accuracy of An_MB_Kb can also be seen in the CFRP bar strain predication. As shown in Fig. 4-18, analytical CFRP strains by the analysis with boundary hoop confinement agree very well with experimental results, while underestimation and overestimation can be observed on tensile strains and compressive strains respectively of the analysis without considering hoop confinement for all specimens.

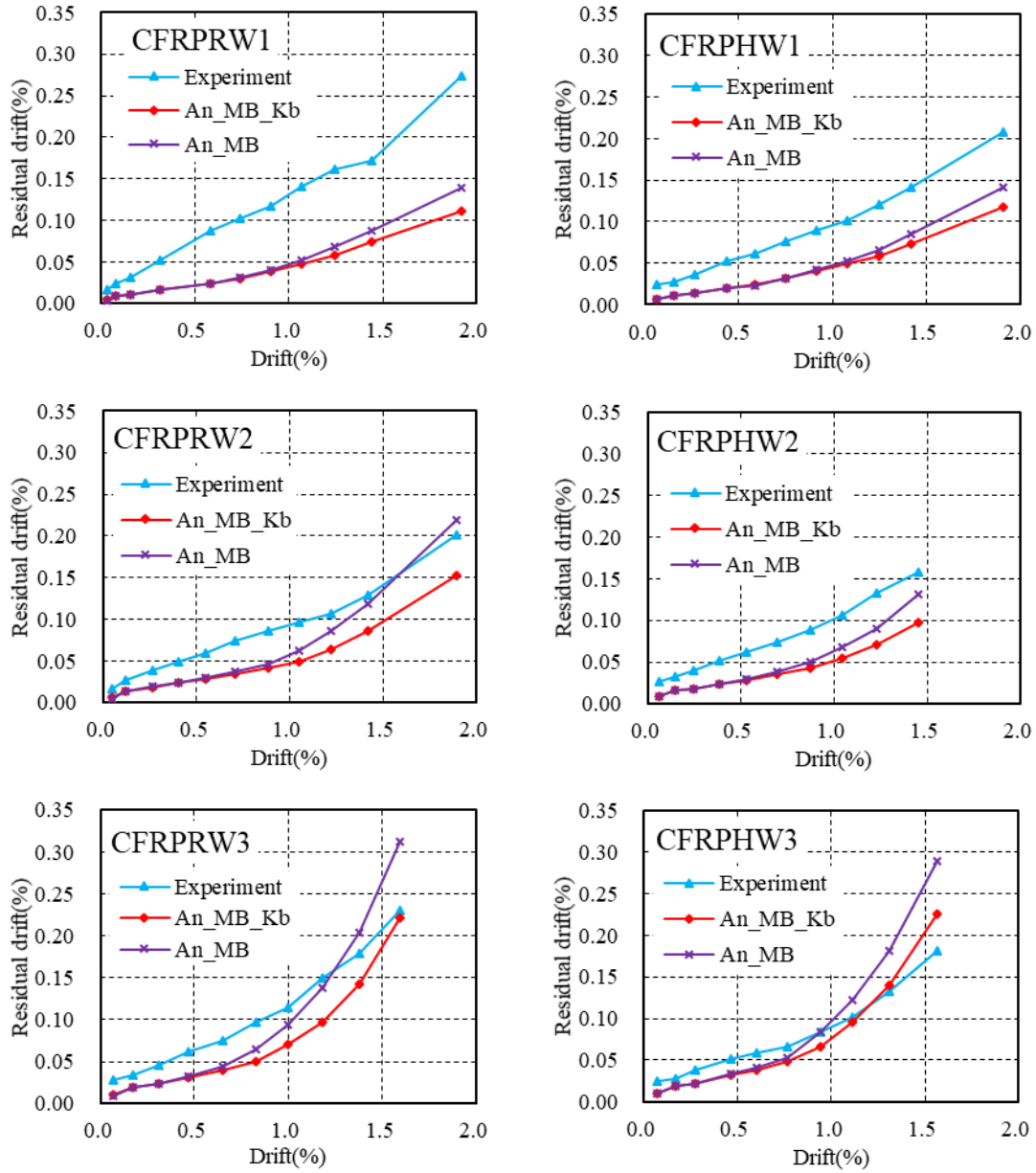


Fig. 4-17 Effect of confinement from boundary element hoops on residual drifts

Where, the legend “Experiment” represents the experimental result, “An_MB” is the analytical result with modified bond strength and corresponding slip, “An_MB_Kb” is the analytical result with modified bond strength, to which corresponding slip, and consideration of confinement from boundary hoops.

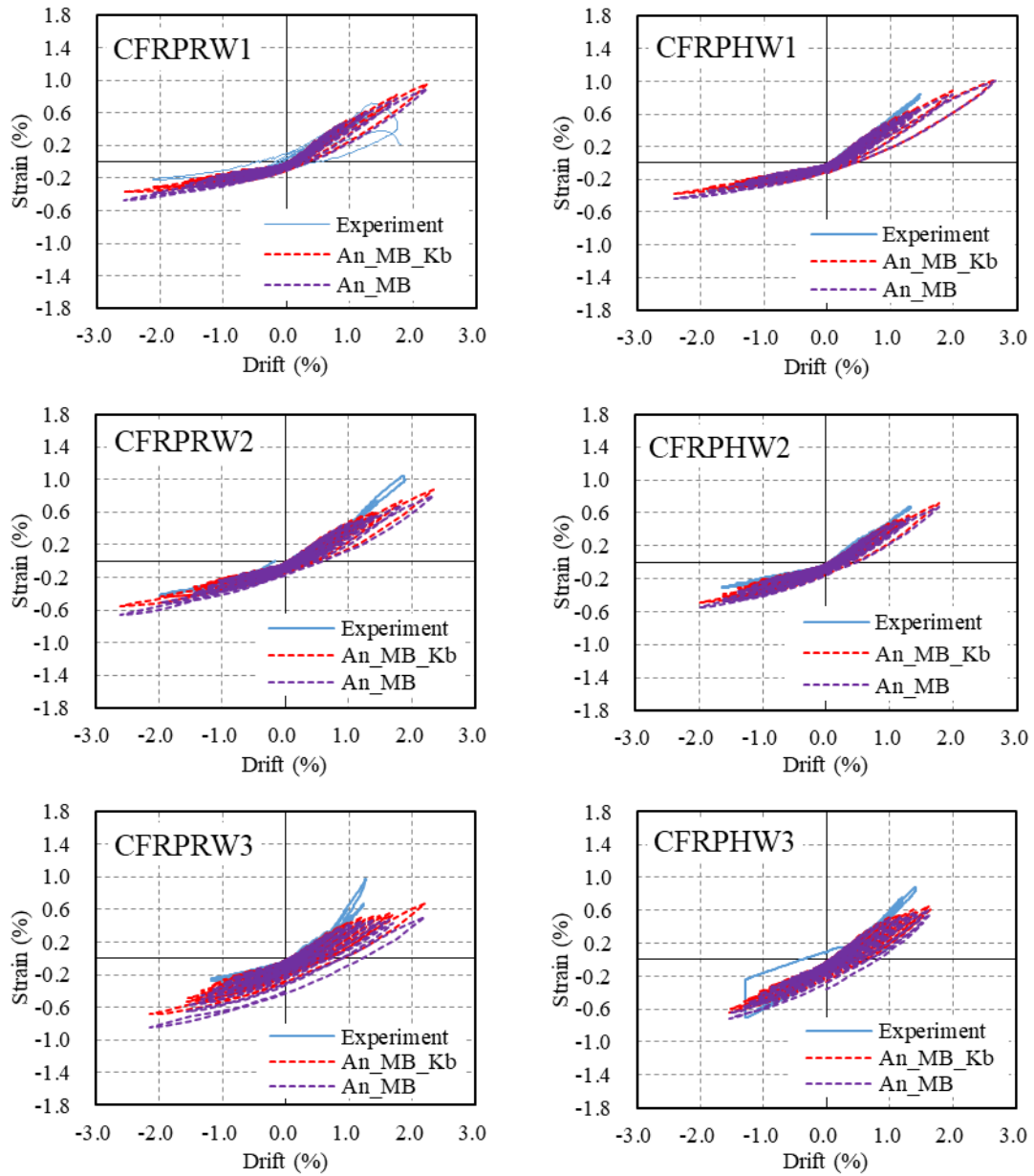


Fig. 4-18 Effect of confinement from boundary element hoops on CFRP bar strains

Where, the legend “Experiment” represents the experimental result, “An_MB” is the analytical result with modified bond strength and corresponding slip, “An_MB_Kb” is the analytical result with modified bond strength, to which corresponding slip, and consideration of confinement from boundary hoops.

4.5 Conclusions

A numerical analytical method was developed in this chapter to assess seismic behavior of concrete walls with their boundary elements being reinforced by CFRP bars and soundly confined by transverse hoops, composite rectangular and circular. The numerical method modified the integrated analytical method by Sun et al. [4.1-4.3] for common concrete components in three aspects; 1) introduction of proper bond strength for CFRP bars based on experimental results available, 2) modification of the slippage corresponding to the peak bond stress for CFRP bars, and 3) consideration of confinement effect by rectangular and/or circular hoops in the boundary elements of wall panel.

Comparisons were conducted between the experimental hysteretic loops with those calculated by the presented analytical method to verify its reliability and accuracy. Based on the comparisons, the following conclusions can be drawn:

- 1) Introducing bond strength and corresponding slip of CFRP bars, the presented analytical method could predict the hysteretic behavior of CFRP bars reinforced concrete walls up to ultimate drift with high and satisfactory accuracy. Besides, the analytical history of steel strains of CFRP bars also traced the experimental results fairly well.
- 2) Ignorance of effects of bond slippage would overestimate the cyclic behaviors of walls, in particular the ultimate lateral resistance. The analytical ultimate lateral resistance was 50% larger than the measured one at maximum.
- 3) Taking confinement effect by transverse hoops in the boundary elements into account could further improve accuracy of the analytical hysteretic curves and CFRP strains of the walls.

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CHAPTER FIVE

5. Evaluation of ultimate capacities of concrete walls reinforced by CFRP bars

5.1 Introduction

As described in chapter four, the overall cyclic behavior of concrete walls with CFRP bars can be predicted satisfactorily well by the analytical methods presented in this thesis. The ultimate lateral capacity, however, is still the most important and direct index used by structural engineers in the practice of design. Therefore, it is important to find or develop appropriate equations to simply and accurately evaluate ultimate capacities of CFRP bars reinforced concrete walls.

There have been many design equations recommended in current design codes and the literatures [5.1-5.3] for ordinary reinforced concrete components. However, it remains unknown if the current equations are applicable to concrete walls reinforced by CFRP bars.

Objective of this chapter is to confirm if the current representative design equations can be used to assess the ultimate lateral capacity of concrete walls reinforced by CFRP bars. The selected evaluating equations are those recommended in Japan (AIJ), the United States (ACI), and China (GB) [5.1-5.3]. Besides the code equations, the so-called NewRC stress block [5.4] is also adopted to evaluate the flexural strength of wall sections with CFRP bars.

5.2 Assessment of ultimate flexural capacity

Eq. (5.1) and Eq. (5.2) are two equations in Japanese code AIJ for assessing flexural strength of reinforced concrete walls. These equations are simplified forms assuming the moment of cross section of wall reaches the maximum value M_u when steel bars yield. For concrete walls reinforced by CFRP bars in boundary elements, the yield strength f_{ys} in Eq. (5.1) and Eq. (5.2) is replaced by ultimate tensile strength during the calculation in this dissertation.

$$M_u = [0.9A_{st}f_{ys} + 0.5A_vf_{yv} + 0.5P(1 - \frac{P}{t_w l_w f'_c})]l_w \quad (5.1)$$

$$M_u = (A_{st}f_{ys} + 0.4A_{sv}f_{sv} + 0.5P) \cdot 0.9l_w \quad (5.2)$$

Where, A_{st} is the area of tensile longitudinal bars the boundary elements, mm^2 , f_{ys} is the yield strength of longitudinal bars in boundary elements, MPa, A_{sv} is the area of all vertical web steel bars, mm^2 , f_{yv} is the yield strength of vertical web bars, MPa, P is the applied axial load, N , t_w is the thickness of the wall panel, mm , l_w is the length (overall depth) of the wall panel, mm , f'_c is the compressive strength of concrete cylinder (200mm height by 100mm diameter), MPa.

The evaluating methods of flexural strength in American and Chinese codes for a given axial load are based on the computing procedure for $M-\phi$ relation of cross section with simplification by replacing the actual stress distribution of concrete in compressive zone with an equivalent rectangular stress block as shown in Fig. 5-1. This method also assumes that the moment resisted by section increases to maximum as the extreme compression concrete fiber achieves ultimate strain ε_{cu} and confinement by transverse reinforcement is neglected. Differences between ACI and GB are calculating equations for the coefficients of equivalent rectangular concrete compressive stress block, α_m and β_m that define ratios of the stress and the depth of equivalent stress block to the compressive concrete stress and the depth of neutral axis, respectively. Besides, the extreme compression strain of concrete ε_{cu} is also different according to ACI and GB codes.

Sun and Sakino [5.4] also proposed calculating equations of α_m , β_m and ε_{cu} called NewRC

to obtain a more accurate predication for flexural strength of reinforced concrete components, which will also be used to verify its applicability on CFRP bars reinforcing concrete walls.

Following ACI code, the maximum usable strain at extreme concrete compression fiber ε_{cu} is assumed to be 0.003. The factor α_m defining the ratio of the stress of equivalent stress block to concrete compressive strength is also a constant of 0.85. The ratio of the depth of stress block to the depth of neutral axis β_m relates to concrete compressive strength, which can be expressed as Eq. (5.3).

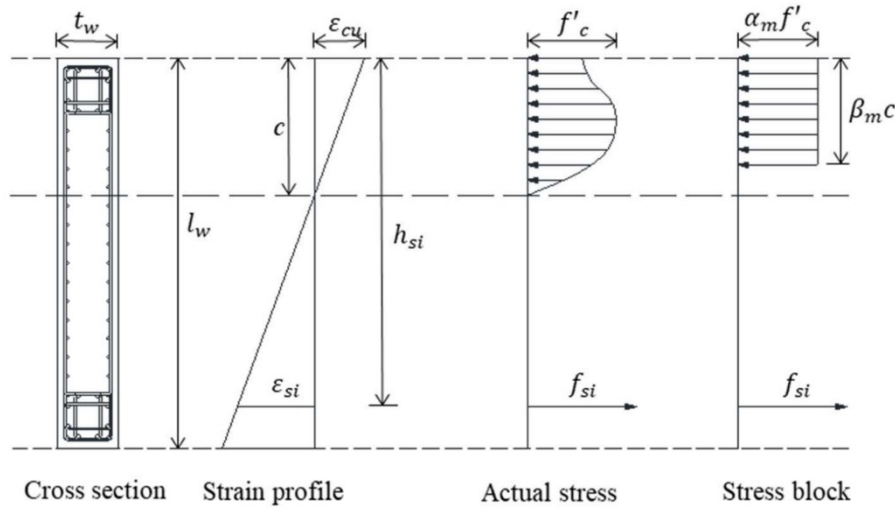


Fig. 5-1 Equivalent stress block of concrete

Where, t_w is the thickness of the wall panel, mm, l_w is the length (overall depth) of the wall panel, mm, c is the depth of neutral axis, mm, ε_{cu} is the maximum strain at extreme concrete compression fiber, h_{si} is the distance from i -th layer of rebars to the extreme compressive fiber, ε_{si} is the strain of i -th layer of rebars, f_{si} is the stress of i -th layer of rebars, MPa, f'_c is the concrete compressive strength, MPa, α_m is the ratio of stress of stress block to concrete compressive strength, β_m is the ratio of the depth of stress block to the depth of neutral axis.

$$\beta_m = \begin{cases} 0.85 & 17.2\text{MPa} \leq f'_c \leq 27.6\text{MPa} \\ 0.85 - \frac{0.2(f'_c - 27.6)}{27.6} & 27.6\text{MPa} < f'_c < 55.2\text{MPa} \\ 0.65 & f'_c \geq 55.2\text{MPa} \end{cases} \quad (5.3)$$

Where, f'_c is the compressive strength of concrete cylinder (300mm height by 150mm diameter), MPa.

The maximum strain at extreme concrete compression fiber ε_{cu} and stress block parameter α_m are not constants as recommended in ACI code, but functions of concrete strength in Chinese code GB 50010. Besides the calculating equation for factor β_m in GB 50010 also differs from that in ACI 318. Eq. (5.4) through Eq. (5.6) express calculating methods for ε_{cu} , α_m and β_m respectively.

$$\varepsilon_{cu} = 0.0033 - (f'_{cu} - 50) \cdot 10^{-5} \leq 0.0033 \quad (5.4)$$

$$\alpha_m = \begin{cases} 1.0 & f'_{cu} \leq 50 \text{ MPa} \\ 1 - \frac{0.1(f'_{cu} - 50)}{50} & 50 \text{ MPa} < f'_{cu} \leq 80 \text{ MPa} \end{cases} \quad (5.5)$$

$$\beta_m = \begin{cases} 0.8 & f'_{cu} \leq 50 \text{ MPa} \\ 0.8 - \frac{0.1(f'_{cu} - 50)}{50} & 50 \text{ MPa} < f'_{cu} \leq 80 \text{ MPa} \end{cases} \quad (5.6)$$

Where, f'_{cu} is the compressive strength of concrete cube (150mm length), MPa.

It is similar to GB 50010 that equations proposed by Sun and Sakino [5.4] for ε_{cu} and β_m are affected by concrete strength as expressed in Eq. (5.7) and Eq. (5.8). Eq. (5.9) is the equation to decide factor α_m that is obtained from f'_c and β_m .

$$\varepsilon_{cu} = [1.483 - 0.1(\frac{f'_c}{42})] \cdot 0.94(f'_c)^{0.25} \cdot 10^{-3} \quad (5.7)$$

$$\beta_m = 0.858 - 0.02(\frac{f'_c}{42}) \quad (5.8)$$

$$\alpha_m = \frac{0.831 - 0.076(\frac{f'_c}{42})}{\beta_m} \quad (5.9)$$

Where, f'_c is the compressive strength of concrete cylinder (200mm height by 100mm diameter), MPa.

Fig. (5.3) through Fig (5.9) express calculating methods of equivalent concrete stress block factors from different codes and literature. It should also be mentioned that the concrete compressive strength f'_c is decided based on different concrete test specimens in

America, China and Japan. For America and Japan, concrete cylinders with respective sizes of 300mm height by 150mm diameter and 200mm height by 100mm diameter are usually adopted to test the compressive strength. While in China, concrete cube with 150mm length is tested to define the basic compressive strength and right square prism with 300mm height by 150mm length is tested to obtain the compressive strength used in structural design. Only the compressive strength of concrete cube (150mm length) f'_{cu} is measured by test in this dissertation, other compressive strengths of different concrete specimens are obtained according to transformational relations in European and Chinese standards [5.3,5.5-5.6]. Table 5-1 shows compressive strengths of several concrete specimens.

Table 5-1 Compressive strength of concrete specimens

Notation		GB	GB	ACI	AIJ & NewRC
Specimen	Shape	Cube	Square prism	Cylinder	Cylinder
	Size	150mm length	300mm height by 150mm length	300mm height by 150mm diameter	200mm height by 100mm diameter
Mark		f'_{cu}	f'_c	f'_c	f'_c
Compressive strength (MPa)		57.2	36.8	46.9	49.4

Hereafter calculating procedures are identical for ACI, GB and NewRC. The strain of each layer of rebars can be obtained by Eq. (5.10) based on the plane-remain-plane assumption once concrete extreme fiber strain ϵ_{cu} is decided.

$$\epsilon_{si} = \frac{c - h_{si}}{c} \epsilon_{cu} \quad (5.10)$$

The axial stresses sustained by steel bars or CFRP bars can be obtained by assuming that their stress-strain relations are typical elastoplastic model and perfect linear-elastic model, respectively. The stress sustained by concrete can be calculated simply through the equivalent stress block without dividing concrete into fibers which do reduce the calculating complexity. Then the axial force N and ultimate M_u (with respect to the central axis of section) sustained by the wall section can be calculated by Eq. (5.11) and Eq. (5.12).

$$N = \alpha f'_c t_w \beta c + \sum A_{si} f_{si} \quad (5.11)$$

$$M_u = \alpha f'_c t_w \beta c \left(\frac{l_w}{2} - \frac{\beta c}{2} \right) + \sum A_{si} f_{si} \left(\frac{l_w}{2} - h_{si} \right) \quad (5.12)$$

Where, A_{si} is the area of i -th layer of rebars, mm^2 .

The depth of neutral axis c in Eq. (5.10) through Eq. (5.12) can be decided by balancing the calculated axial force N by Eq. (5.11) with the applied axial force P through a simple iterative procedure.

After obtaining the ultimate moment sustained by cross section that locates bottom of the wall panel by AIJ, ACI, GB and NewRC, the corresponding flexural capacity V_{fu} can be calculated by Eq. (5.13).

$$V_{fu} = M_u / L \quad (5.13)$$

Table 5-2 Comparison of measured and calculated flexural capacities

Notation	V_{exp} (kN)	$\frac{V_{exp}}{V_{fu,AIJ1}}$	$\frac{V_{exp}}{V_{fu,AIJ2}}$	$\frac{V_{exp}}{V_{fu,ACI}}$	$\frac{V_{exp}}{V_{fu,GB}}$	$\frac{V_{exp}}{V_{fu,NewRC}}$
CFRPRW1	872.13	0.62	0.60	1.22	1.21	1.05
CFRPHW1	818.97	0.58	0.57	1.14	1.13	0.99
CFRPRW2	897.48	0.57	0.55	1.20	1.21	1.05
CFRPHW2	857.29	0.55	0.52	1.15	1.15	1.00
CFRPRW3	922.06	0.55	0.51	1.19	1.21	1.05
CFRPHW3	926.93	0.56	0.52	1.20	1.22	1.06
Average		0.57	0.55	1.18	1.19	1.03
St. Dev.		0.02	0.03	0.03	0.03	0.03

Notes: V_{exp} is the average of measured maximum lateral forces in push and pull directions, $V_{fu,AIJ1}$ and $V_{fu,AIJ2}$ are the calculated ultimate flexural capacities by Eq. (5.1) and Eq. (5.2) in AIJ code, respectively, $V_{fu,ACI}$ is the calculated ultimate flexural capacity by ACI-code provisions, $V_{fu,GB}$ is the calculated ultimate flexural capacity by GB-code equations, $V_{fu,NewRC}$ is the calculated ultimate flexural capacity through equations proposed by Sun and Sakino [5.4].

The ratios of measured maximum lateral capacities to calculated flexural capacities are

shown in Table 5-2. As obvious from the results, flexural capacities ($V_{fu,AIJ1}$ and $V_{fu,AIJ2}$) by AIJ-code overestimate measured experimental capacities (V_{exp}) by over 40% for all specimens because CFRP bars do not achieve ultimate tensile strain in the actual test. While results by ACI and GB code ($V_{fu,ACI}$ and $V_{fu,GB}$) give conservative predications by more than 13% underestimation on measured capacities. Furthermore, one can see from Table 5-2 that the calculated flexural capacities $V_{fu,NewRC}$ agree very well with experimental capacities V_{exp} .

To find the reason for the AIJ equations to overestimate experimental results, Table 5-3 compares the calculated flexural capacities by AIJ equations that utilize the longitudinal strains of CFRP bar obtained from ACI, GB codes and NewRC stress block to replace the ultimate tensile strain of CFRP bar. It can be seen that substituting the CFRP strain obtained by ACI and GB equations into AIJ equation 1 can give a rational predication of flexural capacity. While the calculated results by AIJ equation 2 still overestimate the measured flexural capacities, but the discrepancy has been improved significantly. It can also be seen from Table 5-3 that utilizing the longitudinal strains based on the ACI block enables the AIJ equation 1 to predict the flexural capacity of concrete walls reinforced by CFRP bars with the same accuracy as the NewRC block.

Table 5-3 Calculated flexural capacities by AIJ equations

Notation	$\frac{V_{exp}}{V_{AIJ1,ACI}}$	$\frac{V_{exp}}{V_{AIJ2,ACI}}$	$\frac{V_{exp}}{V_{AIJ1,GB}}$	$\frac{V_{exp}}{V_{AIJ2,GB}}$	$\frac{V_{exp}}{V_{AIJ1,NewRC}}$	$\frac{V_{exp}}{V_{AIJ2,NewRC}}$
CFRPRW1	1.08	1.05	1.05	1.02	0.94	0.91
CFRPHW1	1.02	0.98	0.99	0.96	0.88	0.86
CFRPRW2	1.05	0.96	1.02	0.95	0.93	0.86
CFRPHW2	1.00	0.92	0.98	0.90	0.88	0.82
CFRPRW3	1.02	0.89	1.01	0.88	0.92	0.81
CFRPHW3	1.03	0.90	1.01	0.88	0.93	0.82
Average	1.03	0.95	1.01	0.93	0.91	0.85
St. Dev.	0.03	0.05	0.02	0.05	0.02	0.03

5.3 Assessment of ultimate shear capacity

For ultimate shear capacity, there also two different expressions recommended in AIJ code with consideration of the ratio of boundary longitudinal rebars, steel ratio and yield strength of horizontal web reinforcement, shear span ratio, concrete compressive strength, and axial load ratio as follows:

$$V_{su} = \left[\frac{0.053 \rho_{te}^{0.23} (f'_c + 18)}{\frac{M_w}{V_w l_w} + 0.12} + 0.85 \sqrt{\rho_h f_{yh}} + 0.1 \frac{P}{t_w l_w} \right] t_w \cdot \frac{7}{8} (0.95 l_w) \quad (5.14)$$

$$V_{su} = \left[\frac{0.068 \rho_{te}^{0.23} (f'_c + 18)}{\sqrt{\frac{M_w}{V_w l_w}} + 0.12} + 0.85 \sqrt{\rho_h f_{yh}} + 0.1 \frac{P}{t_w l_w} \right] t_w \cdot \frac{7}{8} (0.95 l_w) \quad (5.15)$$

Where, ρ_{te} is the steel ratio of lumped tensile rebars in the boundary elements in percentage ($=100A_{st}/(0.95t_w l_w)$), f'_c is the compressive strength of concrete cylinder (200mm height by 100mm diameter), $M_w/(V_w l_w)$ is the shear span ratio ($=L/l_w$, L is shear span) ranging between 1.0 and 3.0, ρ_h is the steel ratio of transverse reinforcement in the wall panel, f_{yh} is the yield strength of transverse reinforcement in the wall panel, P is the applied axial load, t_w is the thickness of the wall panel, l_w is the length (overall depth) of the wall panel.

The equation assessing shear capacity in ACI code take into account of effects of the shear span ratio, steel ratio and yield strength of transverse reinforcement expressed as Eq. (5.16). GB code provision Eq. (5.17) for computing ultimate shear capacity considers similar factors with ACI code and ignores the effect of axial load that only appears in Eq. (5.14) and Eq. (5.15) recommended in AIJ code.

$$V_{su} = \left(\frac{1}{12} \alpha_c \sqrt{f'_c} + \rho_h f_{yh} \right) t_w l_w \quad (5.16)$$

Where, the coefficient α_c is 3.0 for shear span ratio L/l_w less than 1.5, is 2.0 for L/l_w larger than 2.0, and varied linearly between 3.0 and 2.0 for L/l_w between 1.5 and 2.0.

$$v_{su} = (\alpha_{cv} f_t + \rho_h f_{yh}) t_w l_{w0} \quad (5.17)$$

Where, l_{w0} is the effective depth of section that can be defined as the distance from center of the boundary element to the outermost edge of another boundary element, the shear capacity coefficient α_{cv} is calculated by Eq. (5.18), f_t is the tensile strength of concrete and can be obtained from compressive strength by Eq. (5.19).

$$\alpha_{cv} = \frac{1.75}{\frac{L}{l_{w0}} + 1} \quad (5.18)$$

Where, L/l_{w0} ranges between 1.5 and 3.0.

$$f_t = 0.88 \times 0.395 \alpha_{c2} f'_{cu} \quad (5.19)$$

Where, α_{c2} can be obtained by Eq. (2.1).

Table 5-3 shows ratios of experimental capacities to calculated shear capacities. As described in chapter two and chapter three that all test walls were dominated by flexure, shear capacities shall be greater than measured lateral capacities. While calculated shear capacities by the first equation recommended in AIJ-code are over 10% smaller than experimental capacities, which indicates that Eq. (5.14) is not suitable to assess ultimate shear capacity for concrete walls reinforced by CFRP bars. As for ACI-code and GB-code equations that do not consider effects of the axial load on shear capacity show conservative evaluations under high axial load ratios. The second equation in AIJ code Eq. (5.15) can give a reasonable assessment for shear capacity of CFRP bars reinforcing walls even under relatively high axial load ratio.

Table 5-4 Comparison of measured and calculated shear capacities

Notation	V_{exp} (kN)	$\frac{V_{exp}}{V_{su,AIJ1}}$	$\frac{V_{exp}}{V_{su,AIJ2}}$	$\frac{V_{exp}}{V_{su,ACI}}$	$\frac{V_{exp}}{V_{su,GB}}$
CFRPRW1	872.13	1.23	0.91	0.91	0.87
CFRPHW1	818.97	1.16	0.86	0.86	0.82
CFRPRW2	897.48	1.15	0.87	0.94	0.90
CFRPHW2	857.29	1.10	0.83	0.90	0.86
CFRPRW3	922.06	1.10	0.85	0.96	0.92
CFRPHW3	926.93	1.10	0.85	0.97	0.93
Average		1.14	0.86	0.92	0.88
St. Dev.		0.05	0.02	0.04	0.04

Notes: V_{exp} is the average of measured maximum lateral forces in push and pull directions, $V_{su,AIJ1}$ and $V_{su,AIJ2}$ are the calculated ultimate shear capacities by Eq. (5.14) and Eq. (5.15) in AIJ code, respectively, $V_{su,ACI}$ is the calculated ultimate shear capacity by ACI-code provisions, $V_{su,GB}$ is the calculated ultimate shear capacity by GB-code equations.

5.4 Conclusions

To find a simple method to predict the ultimate flexural and shear capacities of concrete walls reinforced by CFRP bars, several design equations recommended in AIJ standards (Japan), ACI code (USA), GB code (China) were selected and compared with the experimental results described in chapters two and three. For comparison, the NewRC stress block and equations were also adopted to predict the flexural strength. Based on comparisons between the calculated capacities and the measured results, the following conclusions can be drawn:

- 1) The flexural capacity of concrete walls reinforced by CFRP bars were overestimated by the two simplified equations recommended in AIJ code by 45% on average because these equations use the yield strength of longitudinal bars, while the CFRP bars didn't yield till the end of tests. Substituting the CFRP strain obtained by ACI and GB equations into AIJ code equation 1 gave a rational predication of flexural capacity. The calculated flexural capacities by ACI and GB equations gave a conservative prediction to the test walls by 20% on average.
- 2) The NewRC stress block and equations predicted the flexural capacities very well regardless of axial load level of the wall. The calculated flexural capacities by NewRC equations were only 3% lower than the test results on average.
- 3) Since all the test walls failed due to the rupture of CFRP bars under compression rather than by shear, it is difficult to confirm accuracy of the representative code-prescribed design equations for shear capacity. However, it is noteworthy that all theoretical shear capacities calculated by the equations in AIJ code, ACI code, and GB were larger than the experimental maximum lateral forces. It can also be presumed that all test walls didn't reach their ultimate shear capacities, implying that the three code-prescribed equations would reasonably predict ultimate shear capacity for the concrete walls with CFRP bars. Among the three equations, the AIJ code equation 2 (Eq. (5-15)) might give more rational prediction of shear capacity because it covers more structural factors, in particular the axial load ratio, than the others.

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CHAPTER SIX

6. Modeling of V-R envelop curve of concrete walls reinforced by CFRP bars

6.1 Introduction

The performance-based seismic design method (PBSDM) has been widely recognized in the earthquake engineering community as an essential next step towards resilient and loss-resistant communities. Unlike conventional strength- or stress-based design methodology, which have been adopted in current design codes and standards, the PBSDM utilizes the deformation of building and infrastructures as the design targets, at the levels of strain, curvature, drift, and story drift. Because these structural factors are directly related to the damage degree of structural elements as well as of the whole structures, the PBSDM can provide information about not only the safety under various levels of earthquakes, but the repairable probability of building structures as well.

Fig. 6-1 shows the flowchart that presents the key steps in the performance-based design process [6.1]. The performance objectives in the PBSDM consist of earthquake level and structural performance, which is generally defined in terms of strain, chord rotation, and drift ratio. To assess the performance point of building structures under a given level of earthquake, the envelope curve of lateral force V versus drift R (hereinafter referred to as V-R envelope) is indispensable.

In this chapter, the V-R envelop curve of concrete walls reinforced by CFRP bars will be developed and compared with the experimental results described in chapters two and three.

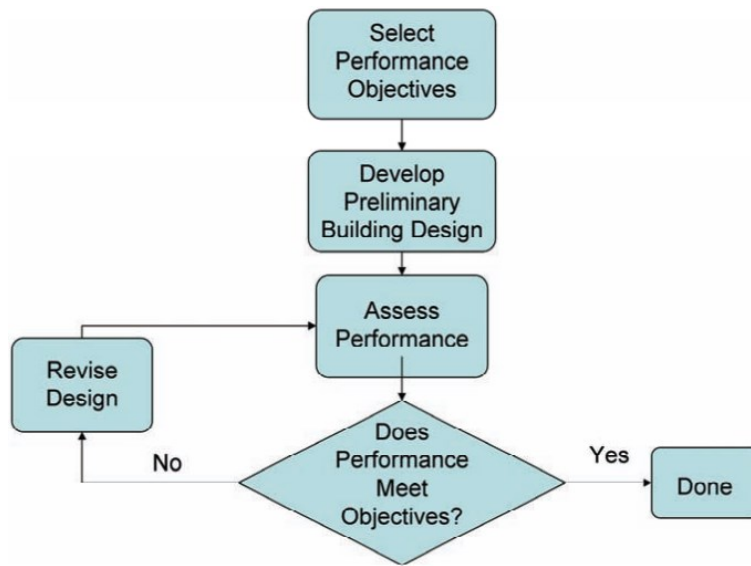


Fig. 6-1 Performance-based design flow diagram [6.1]

6.2 Proposal of V-R envelop curve model

Based on the experimental results described in chapter two and chapter three, the V-R envelop curve for concrete walls reinforced by CFRP bars can be comprised of an ascending part and a descending part as shown in Fig. 6-2. As obvious from Fig. 6-2, information about the peak point (P_1) with maximum lateral force and the ultimate point (P_o) with ultimate drift are two key parameters to define the envelop curve.

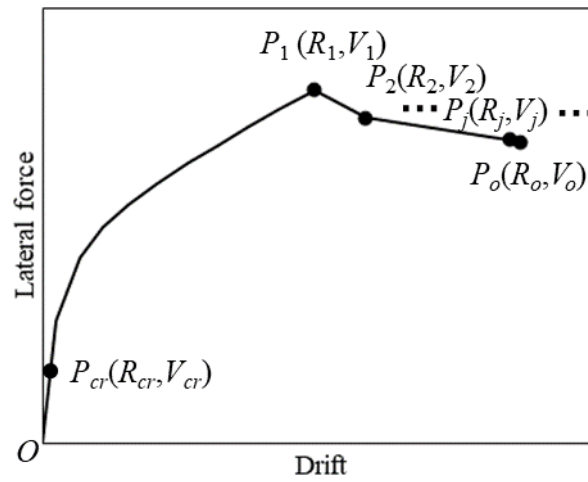


Fig. 6-2 Outline of the proposed capacity envelop curve

Table 6-1 Main formulae in the proposed model

Region		V-R relationship	Critical parameter
Ascending	Initial elastic	$V_i = K_e R_i$ $R_i < R_{cr}$	$K_e = \text{Eq. (2.3)}$ $R_{cr} = \text{Eq. (6.23)}$
	Drift-hardening	$V_i = \left(0.1131 \frac{V_1}{R_1} + 94.315\right) \ln(R_i) + \left(0.5904 \frac{V_1}{R_1} + 377.13\right)$ $R_{cr} \leq R_i \leq R_1$	$R_1 = \text{Eq. (6.6)}$ $V_1 = \text{Eq. (6.10)}$
Descending		Multi-linear lines connected by points of $P_1(R_1, V_1)$, $P_2(R_2, V_2)$, $P_j(R_j, V_j)$, $\dots$, $P_o(R_o, V_o)$ $R_i > R_1$	$R_j = \text{Eq. (6.15)}$ $V_j = \text{Eq. (6.18)}$

In developing of the V-R model, it is necessary to obtain the peak point and ultimate point

first to clarify the transition point from the ascending portion to the descending branches and the ending point of the envelop curve. Table 6-1 summarizes mathematic expressions for the proposed V-R envelop curve model.

6.2.1 The peak lateral force and corresponding drift

According to experimental results of concrete walls reinforced by CFRP bars, degradation of lateral force is attributed to compression failure of CFRP bars. The peak lateral force, therefore, is reached when the outermost CFRP bar in compressive boundary element (the first layer of CFRP bars from the compressive side of section) sustains extreme compression strain ε_{CFRPcu} (see Fig. 6-3).

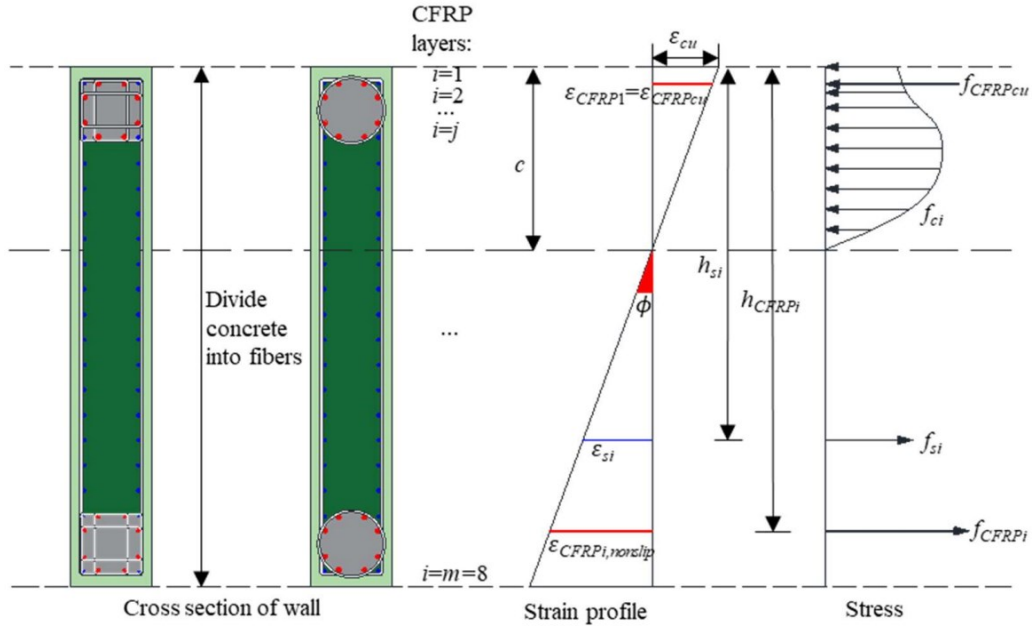


Fig. 6-3 Strain and stress distributions in a concrete wall section

Then curvature of the cross section ϕ can be calculated in the form of:

$$\phi = \frac{\varepsilon_{CFRPcu}}{c - h_{CFRP1}} \quad (6.1)$$

Based on the plane-remain-plane assumption, concrete extreme fiber strain ε_{cu} can be expressed as:

$$\varepsilon_{cu} = c\phi \quad (6.2)$$

And strains of i -th concrete fiber ε_{ci} , i -th layer of steel bars ε_{si} and i -th layer of CFRP bars (without slip) $\varepsilon_{CFRPi,nonslip}$ are calculated by the flowing equation:

$$\varepsilon_{ci} = \frac{(c-h_{ci})}{c} \varepsilon_{cu}, \quad \varepsilon_{si} = \frac{(c-h_{si})}{c} \varepsilon_{cu}, \quad \varepsilon_{CFRPi,nonslip} = \frac{(c-h_{CFRPi})}{c} \varepsilon_{cu} \quad (6.3)$$

As described in chapter four, CFRP bar strains will be overestimated without considering slippage of CFRP bars. Onaka proposed a simplified method to take into account of the slip effect by multiplying strain of rebar without slip by a reduction factor expressed as Eq. (6.4) [6.2]. That method is based on the simplified concentrated hinge model (see Fig. 4-1) and apportions rebar strain in hinge zone among whole region consists of hinge zone and bond-slip zones uniformly (see Fig. 6-4).

$$\varepsilon_{slip} = \frac{L_p}{L_p + L_b + L_c} \varepsilon_{nonslip} \quad (6.4)$$

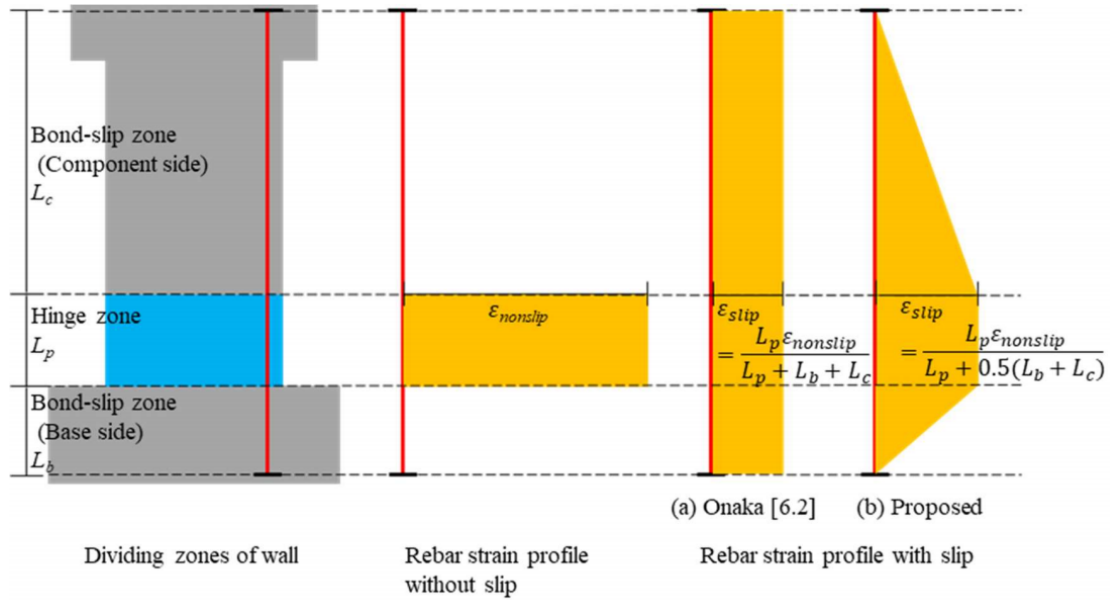


Fig. 6-4 Strain profile of rebar with consideration of slip effect

While for CFRP bars, the measured tensile strain distributions described in chapter two and chapter three exhibit the phenomenon that tensile strain distributed uniformly within

hinge zone and decreased along CFRP bar to anchoring end (see Fig. 2-17, Fig. 3-10). Hence the simplified strain profile of CFRP bar for calculation is a tri-linear model as shown in Fig. 6-4. And in order to further simplify calculating procedure, the slip effect is considered only when CFRP bars sustain tensile strain while slippage of CFRP bars will be neglected under compression. Based on above assumptions, CFRP bar strain with slip can be obtained by the following equation:

$$\varepsilon_{CFRP,slip} = \begin{cases} \frac{L_p}{L_p + 0.5(L_b + L_c)} \varepsilon_{CFRP,nonslip} & \varepsilon_{CFRP,nonslip} < 0 \\ \varepsilon_{CFRP,nonslip} & \varepsilon_{CFRP,nonslip} > 0 \end{cases} \quad (6.5)$$

Where, strain is positive for compression and negative for tension.

The axial stresses sustained by concrete fiber (f_{ci}), steel bar (f_{si}) and CFRP bar (f_{CFRPi}) can be obtained by their stress-strain relations described in chapter four. Before calculating the peak lateral force, the corresponding drift should be obtained first to take into consideration of P-Δ effect. Because the same simplified concentrated hinge model as numerical analytical method in chapter four is adopted (see Fig. 4-1), therefore drift can be obtained from section curvature by Eq. (6.6) through (6.7).

$$R = R_f / (1 - \alpha) \quad (6.6)$$

$$R_f = \phi L_p (1 - \frac{L_p}{2L}) \quad (6.7)$$

Where, R_f is the drift caused by flexural deformation, α is the ratio of shear drift to overall drift that is calculated by procedures described in 4.2.6, ϕ is the curvature of section within hinge zone, L is the shear span.

After obtaining drift R_1 , the axial force N_1 and moment M_1 and lateral force V_1 can be calculated by Eq. (6.8) through Eq. (6.10).

$$N_1 = \sum_{i=1}^m A_{CFRPi} f_{CFRPi} + \sum_{i=1}^h A_{si} f_{si} + \sum_{i=1}^k A_{ci} f_{ci} \quad (6.8)$$

$$M_1 = \sum_{i=1}^m A_{CFRPi} f_{CFRPi} \left(\frac{l_w}{2} - h_{CFRPi} \right) + \sum_{i=1}^h A_{si} f_{si} \left(\frac{l_w}{2} - h_{si} \right) + \sum_{i=1}^k A_{ci} f_{ci} \left(\frac{l_w}{2} - h_{ci} \right) \quad (6.9)$$

$$V_1 = \frac{M_1}{L} - PR_{f1} \quad (6.10)$$

Where, A_{CFRPi} , A_{si} , A_{ci} are the corresponding areas to f_{CFRPi} , f_{si} , f_{ci} , h_{ci} is the distance from center of i -th concrete fiber to the extreme compressive fiber, m and h are the numbers of layers of CFRP bars and steel bars respectively, k is the number of concrete fibers.

6.2.2 The descending part of envelop curve

The descending part of envelop curve is defined by several points ($P_1, P_2, \dots, P_j, \dots, P_o$) with which straight lines are used to connect as shown in Fig. 6-2. Coordinates of P_1 have been calculated in 6.2.1. The ending point P_o with the ultimate drift will be decided in the follow section 6.2.3. Other control points ($P_2, \dots, P_j$) for the descending part of envelop are calculated here.

$P_2 (R_2, V_2)$ is decided based on assumptions that the second layer of CFRP bars from the extreme compressive fiber reaches its ultimate compressive strain ε_{CFRPcu} and meanwhile the first layer of CFRP bars and the m -th layer of CFRP bars (the first layer from extreme tensile concrete fiber) from the extreme compressive concrete fiber both quit sustaining axial stress because of compression failure. According to that assumptions, curvature of section can be calculated by Eq. (6.11). Strains and stresses of consisting elements of the wall section (ε_{ci} , ε_{si} , ε_{CFRPi} , f_{ci} , f_{si} , f_{CFRPi}) and drift R_2 can be obtained using the same method as described in 6.2.1 (see Eq. (6.2) through Eq. (6.7)).

$$\phi = \frac{\varepsilon_{CFRPcu}}{c - h_{CFRP2}} \quad (6.11)$$

Then the lateral force of point P_2 , V_2 can be obtained using equations below:

$$N_2 = \sum_{i=2}^{m-1} A_{CFRPi} f_{CFRPi} + \sum_{i=1}^h A_{si} f_{si} + \sum_{i=1}^k A_{ci} f_{ci} \quad (6.12)$$

$$M_2 = \sum_{i=2}^{m-1} A_{CFRPi} f_{CFRPi} \left(\frac{l_w}{2} - h_{CFRPi} \right) + \sum_{i=1}^h A_{si} f_{si} \left(\frac{l_w}{2} - h_{si} \right) + \sum_{i=1}^k A_{ci} f_{ci} \left(\frac{l_w}{2} - h_{ci} \right) \quad (6.13)$$

$$V_2 = \frac{M_2}{L} - PR_{f2} \quad (6.14)$$

Where, m is the total number of layers of CFRP bars placed in wall section, h is the total number of layers of steel bars, k is the total number of concrete fibers.

For j -th point of the descending part of envelop curve ($j \leq 0.5m$), the corresponding curvature of section ϕ can be obtained by Eq. (6.15) based on the assumption that strain of j -th layer of CFRP bars from the compressive side reaches ultimate value ε_{CFRPcu} when CFRP bars fail except for layers from j -th to $(m-j+1)$ -th.

$$\phi = \frac{\varepsilon_{CFRPcu}}{c - h_{CFRPj}} \quad (6.15)$$

After obtaining ε_{ci} , ε_{si} , ε_{CFRPi} , f_{ci} , f_{si} , f_{CFRPi} and R_j by Eq. (6.2) through (6.7), V_j can be calculated in the form of:

$$N_j = \sum_{i=j}^{m-j+1} A_{CFRPi} f_{CFRPi} + \sum_{i=1}^h A_{si} f_{si} + \sum_{i=1}^k A_{ci} f_{ci} \quad (6.16)$$

$$M_j = \sum_{i=j}^{m-j+1} A_{CFRPi} f_{CFRPi} \left(\frac{l_w}{2} - h_{CFRPi} \right) + \sum_{i=1}^h A_{si} f_{si} \left(\frac{l_w}{2} - h_{si} \right) + \sum_{i=1}^k A_{ci} f_{ci} \left(\frac{l_w}{2} - h_{ci} \right) \quad (6.17)$$

$$V_j = \frac{M_j}{L} - PR_{fj} \quad (6.18)$$

6.2.3 The ultimate drift and corresponding lateral force

There are two cases need to be compared to decide the ending point $P_o (R_o, V_o)$ with ultimate drift of capacity curve. Case A is the innermost layer of CFRP bars sustained compression reaches maximum compression strain, in other words, j in Eq. (6.15) through Eq. (6.18) equals $0.5m$ that means all CFRP bars in the compressive boundary element achieve or exceed maximum compression limit. The ending point in case A is $P_{0.5m} (R_{0.5m}, V_{0.5m})$. Case B is that the lateral force declines to $0.85V_1$ that is an ending value recommended by Chinese standard [6.3]. The corresponding drift R_G to $0.85V_1$ can be obtained by calculating the ordinate of the intersection of descending capacity curve and horizontal line of $V=0.85V_1$. The ending point in case B is marked as $P_G (R_G, 0.85V_1)$. The maximum between $V_{0.5m}$ and $0.85V_1$ and the corresponding drift are selected as coordinates of ending point of envelop curve, which can be expressed as:

$$V_o = \max(V_{0.5m}, 0.85V_1), R_o = \begin{cases} R_{0.5m} & V_o = V_{0.5m} \\ R_G & V_o = 0.85V_1 \end{cases} \quad (6.19)$$

6.2.4 The ultimate compressive strain of CFRP bar

As described above, it is necessary to confirm the ultimate compressive strain of CFRP bar ε_{CFRPcu} before calculating the capacity curve of wall. Table 6-2 presents ultimate compressive strains of CFRP bars in tests and analytical ultimate compressive strains using numerical analytical method in chapter four. It should be mentioned that the numerical analytical method cannot consider compression failure of CFRP bars, then analytical ultimate compressive strains of CFRP bars are obtained corresponding to the drifts where maximum compressive strains of CFRP bars are measured in experiments.

Table 6-2 Ultimate compressive strains of CFRP bars

Notation	CFRPRW 1	CFRPHW 1	CFRPRW 2	CFRPHW 2	CFRPRW 3	CFRPHW 3
Measured ε_{CFRPcu} (%)	0.44930	0.46360	0.46745	0.48955	0.45400	0.43060
Analytical ε_{CFRPcu} (%)	0.38845	0.38170	0.45577	0.43300	0.44357	0.47067

As indicated in Table 6-2, $\varepsilon_{CFRP_{cu}}$ vary from 0.38% to 0.49%. In the following calculation for V-R envelop curves, 0.40%, 0.45% and 0.50% are adopted as value of $\varepsilon_{CFRP_{cu}}$.

To better compare the effect of maximum compressive strain of CFRP bar $\varepsilon_{CFRP_{cu}}$ on capacity curves, coordinates of the peak point $P_1 (V_1, R_1)$ and ultimate point $P_o (V_o, R_o)$ obtained by the proposed model using different $\varepsilon_{CFRP_{cu}}$ are compared with measured experimental results in detail as shown in Fig. 6-5. In general, accuracy is improved with increasing $\varepsilon_{CFRP_{cu}}$, the proposed envelop curve model can give good predications for not only peak lateral force but also terminate lateral force for all specimens with different hoops and axial load ratios when $\varepsilon_{CFRP_{cu}}$ equals 0.005. As for deformation of walls, it can also obtain reasonable calculated results to use the proposed model with the assumption of taking the ultimate compressive strain of CFRP bar as 0.005.

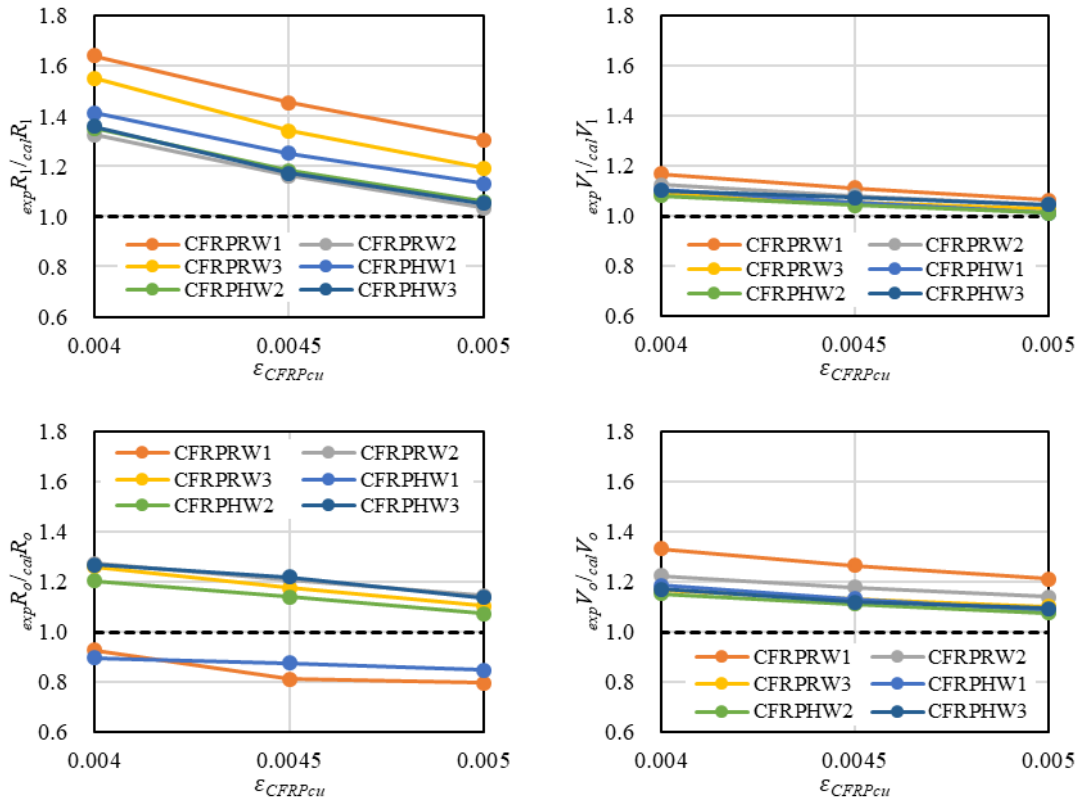


Fig. 6-5 Comparison of calculated coordinates of specific points with experimental results

Where, $expR_1$, $expV_1$, $expR_o$, $expV_o$ are the experimental lateral forces and corresponding drifts

of peak point and terminate point, respectively; $_{cal}R_1$, $_{cal}V_1$, $_{cal}R_o$, $_{cal}V_o$ are the calculated lateral forces and corresponding drifts of peak point and terminate point by proposed envelop curve model, respectively.

6.2.5 The ascending part of envelop curve

The ascending part of envelop curve consists of initial elastic region (OP_{cr}) and drift-hardening region ($P_{cr}P_1$), as shown in Fig. 6-2. The initial elastic region can be determined through the initial stiffness K_e (see Eq. (2.3) through (2.6)) and cracking lateral force V_{cr} . The cracking lateral force V_{cr} means the lateral force which flexural crack first occurs, which can be predicted by the following equations recommended by ACI-318 code [6.4].

$$V_{cr} = M_{cr}/L \quad (6.20)$$

$$M_{cr} = \frac{f_r I_g}{y_t} \quad (6.21)$$

$$f_r = 0.62\lambda\sqrt{f'_c} \quad (6.22)$$

Where, M_{cr} is the moment that causes flexural crack, L is the shear span, mm, I_g is the moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4 , f_r is the modulus of rupture of concrete, MPa, f'_c is the concrete compressive strength, MPa, y_t is the distance from centroidal axis of gross section, neglecting reinforcement, to tension face, mm, λ is the modification factor reflecting the reduced mechanical properties of lightweight concrete and is taken as unit for normal-weight concrete.

After K_e and V_{cr} are determined by Eq. (2.3) and Eq. (6.20), the drift corresponding to V_{cr} , R_{cr} can be simply calculated by dividing V_{cr} with the K_e .

$$R_{cr} = V_{cr}/K_e \quad (6.23)$$

For the drift-hardening region ($P_{cr}P_1$), it is needless to consider the compression failure of CFRP bar. Unlike descending part that curvature of section in hinge zone is calculated according to the maximum compressive strain of CFRP bar, section curvature ϕ within the ascending curve can be calculated simply based the given drift R ($<R_1$) by Eq. (6.24) and Eq. (6.25).

$$R_f = (1 - \alpha)R, R_{cr} < R < R_1 \quad (6.24)$$

$$\phi = \frac{R_f}{L_p(1 - \frac{L_p}{2L})} \quad (6.25)$$

After obtaining section curvature ϕ , strains and stresses of concrete fibers and rebars can be calculated by Eq. (6.2) through Eq. (6.5) and material models in chapter four, respectively. Finally, the corresponding lateral force V to the given R is obtained as

$$M = \sum_{i=1}^m A_{CFRPi} f_{CFRPi} + \sum_{i=1}^h A_{si} f_{si} + \sum_{i=1}^k A_{ci} f_{ci} \quad (6.26)$$

$$M = \sum_{i=1}^m A_{CFRPi} f_{CFRPi} (\frac{l_w}{2} - h_{CFRPi}) + \sum_{i=1}^h A_{si} f_{si} (\frac{l_w}{2} - h_{si}) + \sum_{i=1}^k A_{ci} f_{ci} (\frac{l_w}{2} - h_{ci}) \quad (6.27)$$

$$V = \frac{M}{L} - PR_f \quad (6.28)$$

Fig. 6-6 shows relationships between the calculated lateral force, using M- ϕ analyzing procedures based on Eq. (6.24) through Eq. (6.28), and drift within drift-hardening region ($P_{cr}P_1$) for all specimens. As shown in Fig. 6-6, the following logarithmic equation is proposed to predict the lateral force:

$$V_i = A \ln(R_i) + B \quad (6.29)$$

Apparently, the coefficients A and B for the logarithmic function (6.29) shown in Fig. 6-6 both increase with increasing the peak lateral force V_1 , while decreasing the drift of peak point R_1 (V_1 and R_1 are obtained by Eq. (6.10) and (6.6) in section 6.2.1). Therefore, the combined factor V_1/R_1 is introduced to express the formula for two coefficients A and B . Fig. 6-7 displays the relationships between coefficient A (and B) in Eq. (6.29) and factor V_1/R_1 . It is clear that two coefficients (A and B) strongly correlate with the factor V_1/R_1 . From Fig. 6-7, it is easy to derive expressions for A and B , and the drift-hardening region curve $P_{cr}P_1$ can be defined by Eq. (6.30).

$$V_i = \left(0.1131 \frac{V_1}{R_1} + 94.315\right) \ln(R_i) + \left(0.5904 \frac{V_1}{R_1} + 377.13\right) \quad (6.30)$$

Where, V_i , V_1 are in kN, R_i , R_1 are in %.

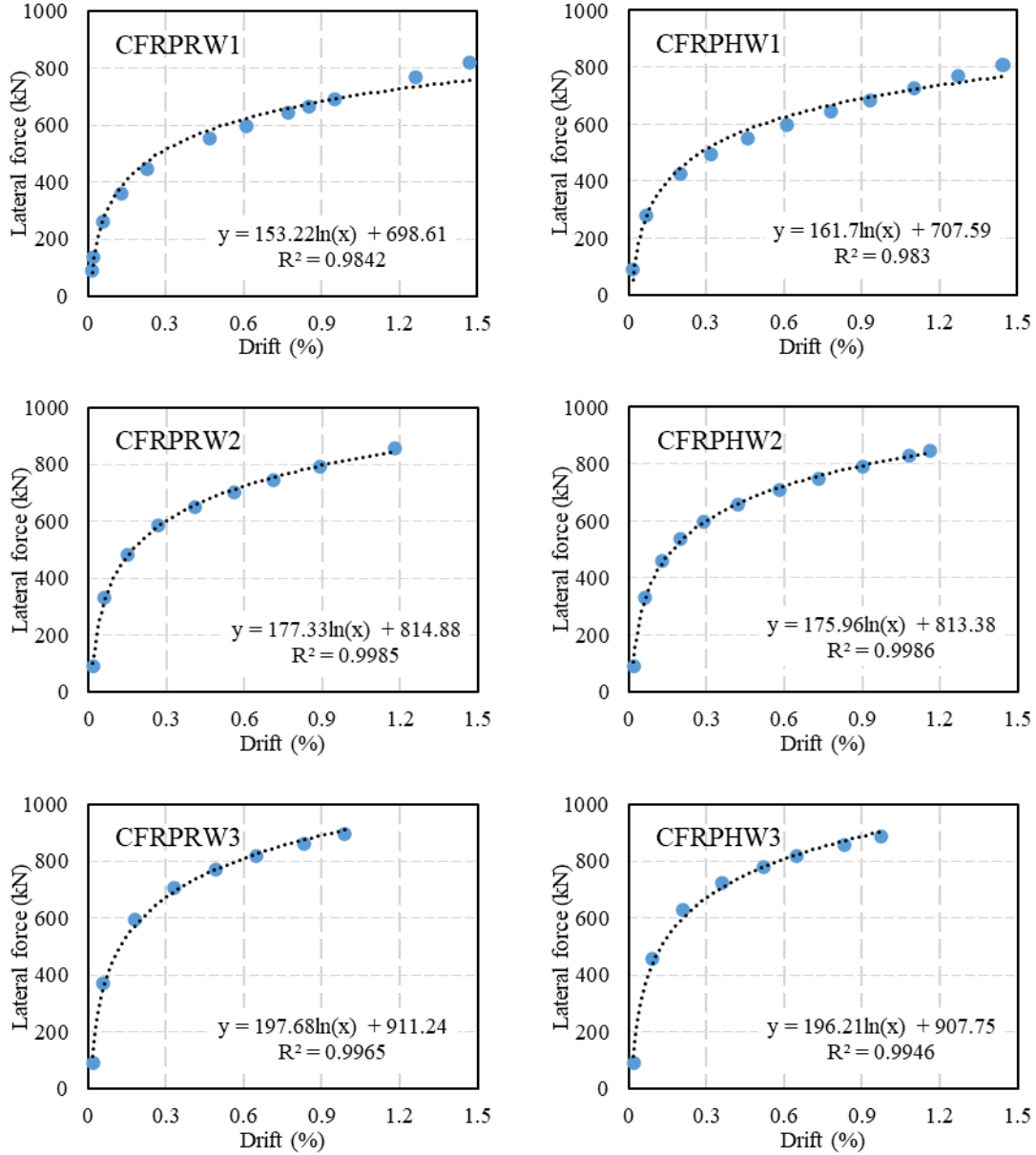
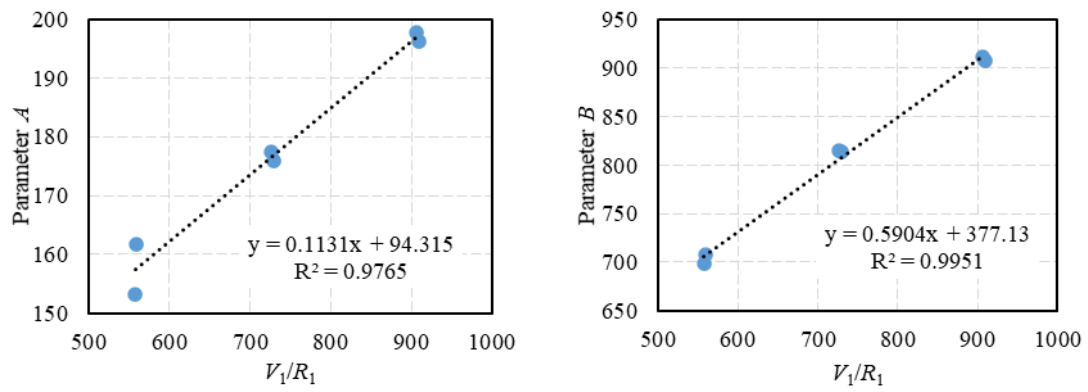


Fig. 6-6 Relationships between calculated lateral force and drift

Fig. 6-7 Relationships between basic coefficients and V_1/R_1

6.3 Verification of the proposed model

In order to verify accuracy of the proposed V-R envelop curve model, the experimental cyclic curves described in chapter two and chapter three are compared with calculated capacity curves in Fig. 6-8. One can see from Fig. 6-8 that the proposed V-R envelop curve model can give a reasonable evaluation of the overall lateral resistance capacity to concrete walls reinforced by CFRP bars.

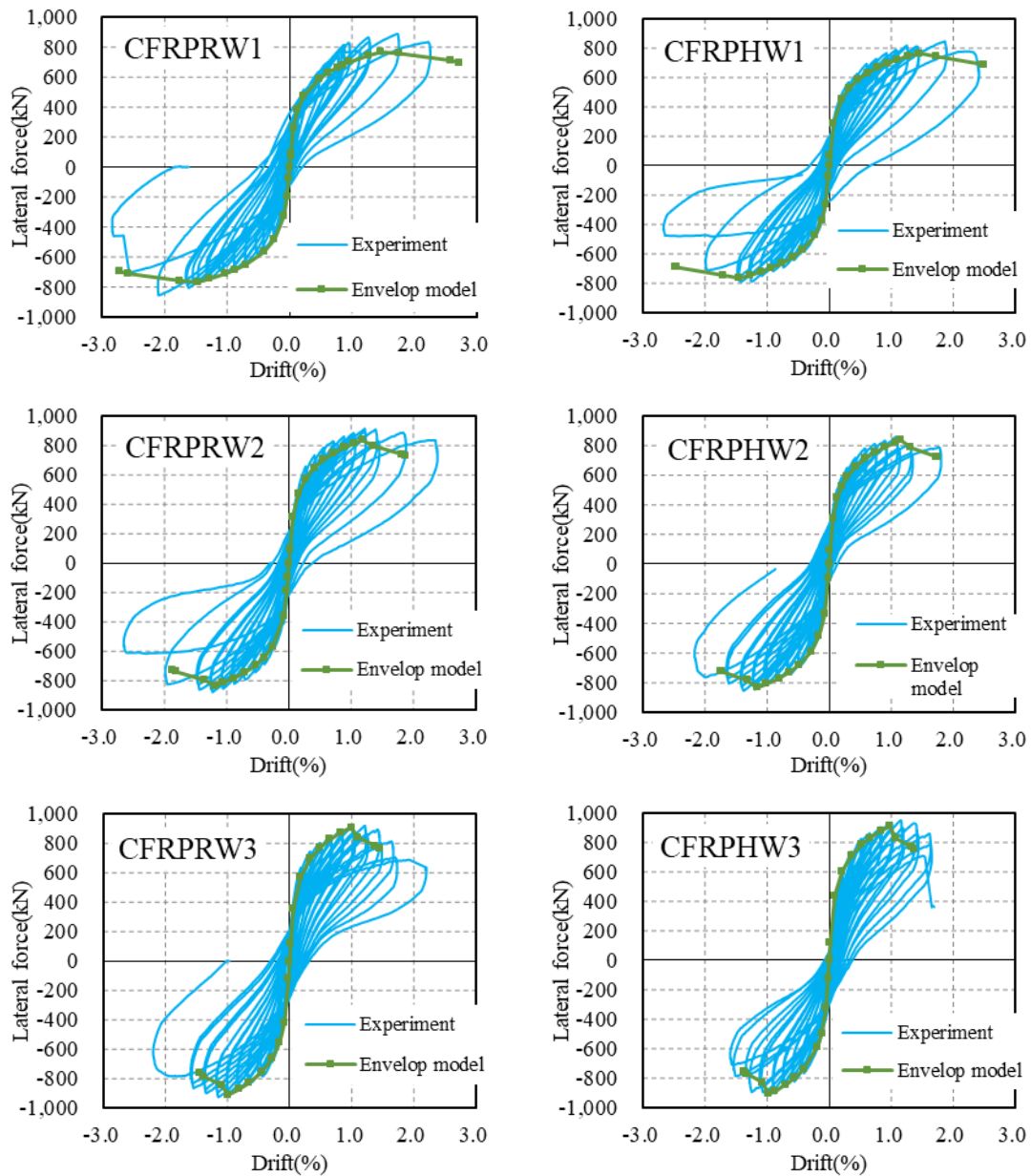


Fig. 6-8 Comparison of calculated envelop curves with measured V-R responses

Fig. 6-9 shows comparisons of envelop curves by the proposed model and hysteretic curves by the numerical analytical method described in chapter four. It can be seen that the envelop curve model exhibits almost the same precision as the numerical analysis that needs complicate procedures to consider effects of slippage before the peak point, which indicates that using the proposed envelop curve model has advantages of both accuracy and simplicity that can provide design references for practical engineering.

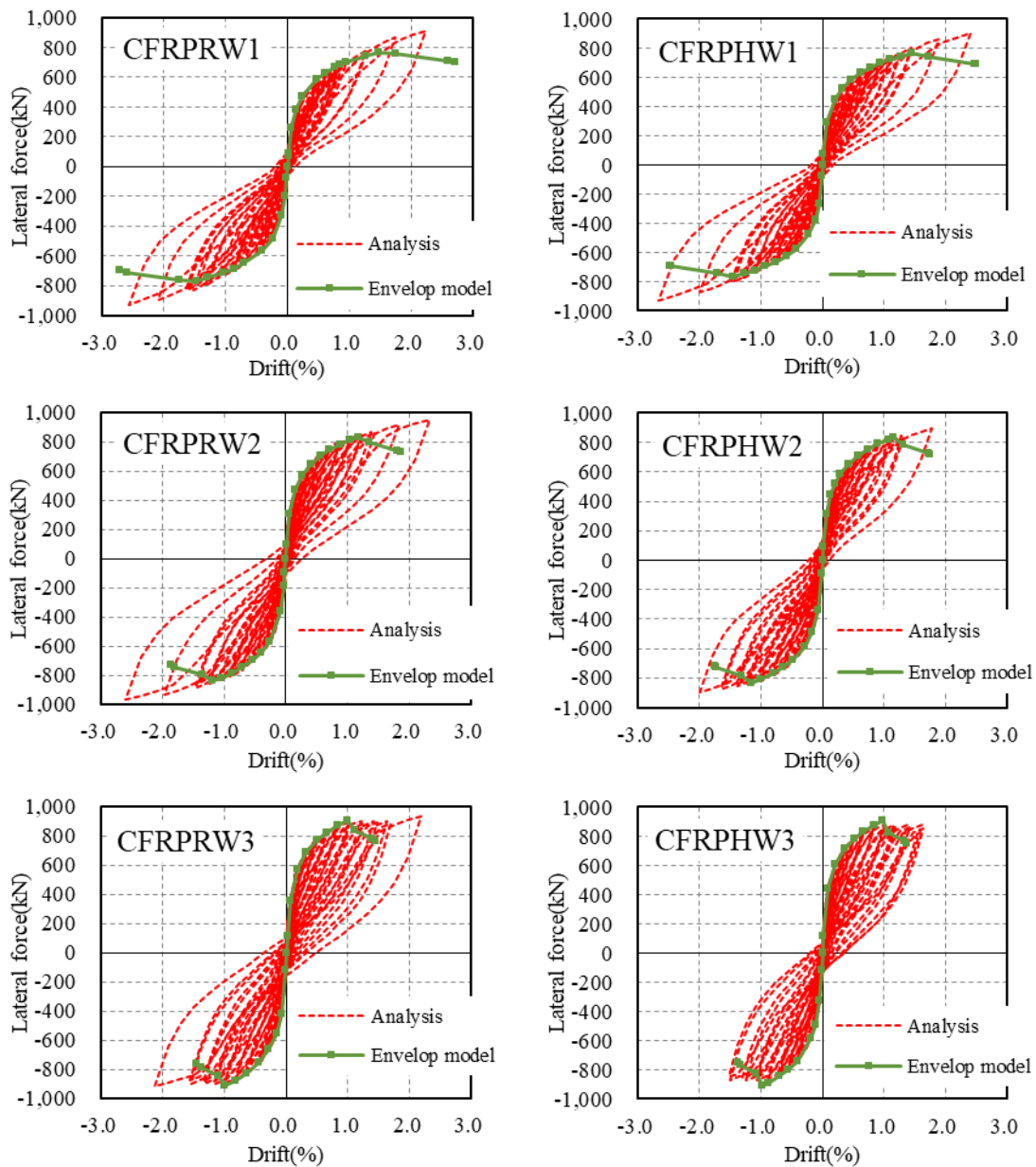


Fig. 6-9 Comparison of proposed envelop curve model with numerical analysis

6.4 Conclusions

In this chapter, a simplified capacity curve model with no need for complicated programming has been developed to facilitate performance-based seismic design of concrete walls reinforced by CFRP bars.

Through comparisons between the experimental results described in chapters two and three and the numerical predictions calculated by the proposed model, the following conclusions can be drawn:

- 1) Taking the experimentally measured compressive strain of 0.005 as the rupture strain for the compressive CFRP bars located at the extreme fiber of wall section could give a fairly good prediction of the peak lateral resistance and the corresponding drift for concrete walls reinforced with CFRP bars.
- 2) The ascending portions of the proposed V-R model agreed very well with the experimental results till the peak points. The descending portion also exhibited good agreement with the measured envelope curves, but conservatively underestimated the ultimate drift for the walls under higher axial load ratios. This might be attributed that the strain of 0.005 underestimated the rupture strain for the CFRP bars located at the inner layer, where the restraint from the surrounding non-damaged concrete may enhance the rupture strain of the CFRP bars.

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CHAPTER SEVEN

7. Conclusions and future works

7.1 Conclusions

Experimental and numerical studies were conducted to investigate to what extent (drift level) the utilization of carbon fiber reinforced plastic (CFRP) bars as longitudinal rebars could make concrete walls resilient. Objectives of this thesis focus on three items; they are 1) seismic behavior of concrete walls reinforced by CFRP bars, 2) seismic behavior of the walls with improved confinement in the boundary elements, and 3) Assessing methods of CFRP bars reinforced concrete walls.

To figure out the issues mentioned above, experimental and analytical studies are conducted and described in detail in chapter two through chapter six.

As the conclusions of this thesis, main findings achieved from the experimental and analytical works described in chapter two through chapter six are summarized as below.

In chapters two and three, six full-scale CFRP bar reinforcing concrete walls with rectilinear or circular hoops in boundary elements were fabricated and tested under cyclic reversed lateral load and axial compression with three axial load ratios of 0.17, 0.26 and 0.33. From the experimental results and discussion, the following conclusions can be drawn:

- 1) The ultimate state (peak point at the V-R curve) of concrete walls reinforced by CFRP bars was dominated by the rupture of the CFRP bars under compression. With the increment of axial load ratio, the stable drift-hardening capacity of concrete walls

decreased from 2.0% drift to 1.2% drift due to the compression failure of CFRP bars and the crushing of concrete shell of the walls.

- 2) The residual drift ratio of the wall was kept less than 0.3% after unloading from the 1.5% drift level regardless of axial load level due to the long elasticity of CFRP bars. However, to ensure the walls a drift-hardening capacity of 2.0%, which has been defined as the drift of life safety for common concrete components in current design codes, the axial load ratio should be limited below 0.25 for concrete walls reinforced with CFRP bars.
- 3) The measured minimum equivalent viscous damping for the six test walls was beyond 0.095, implying that concrete walls reinforced by CFRP bars had higher energy-dissipation capacity than UPTP walls.
- 4) Circular hoops confining concrete walls reinforced by CFRP bars can achieve equivalent seismic behavior as compared with composite rectilinear hoops confining walls with a 64% reduction of steel amount and improvement of construction.

Chapter four through chapter six presented methods of evaluating the seismic behavior of concrete walls reinforced by CFRP bars, including a numerical analytical method for hysteresis behavior, code equations for ultimate capacities and a V-R envelop curve model. Comparisons were conducted between the experimental results with those calculated by the presented assessing methods. Based on the comparisons, the following conclusions can be drawn:

- 1) Introducing bond strength and corresponding slip of CFRP bars, the presented analytical method could predict the hysteretic behavior of CFRP bars reinforced concrete walls up to ultimate drift with high and satisfactory accuracy. Besides, the analytical history of steel strains of CFRP bars also traced the experimental results fairly well.
- 2) Taking confinement effect by transverse hoops in the boundary elements into account could further improve accuracy of the analytical hysteretic curves and CFRP strains of the walls.

- 3) The NewRC stress block and equations predicted the flexural capacities very well regardless of axial load level of the wall. The calculated flexural capacities by NewRC equations were only 3% lower than the test results on average.
- 4) The AIJ code equation 2 (Eq. (5-15)) might give more rational prediction of shear capacity because it covers more structural factors, in particular the axial load ratio, than the other code equations.
- 5) The ascending portions of the proposed V-R model agreed very well with the experimental results till the peak points. The descending portion also exhibited good agreement with the measured envelope curves, but conservatively underestimated the ultimate drift for the walls under higher axial ratios.

7.2 Future works

This dissertation has investigated seismic behavior and proposed methods for predicting cyclic behavior and ultimate capacities of concrete walls reinforced by CFRP bars with the variables of hoop configuration in the boundary elements and the axial load ratios. There are still several issues that have not been addressed yet due to time-constraint. Those unsolved issues are listed below as the future works to promote and amplify the application of CFRP bars reinforcing concrete walls.

- 1) Seismic behavior of concrete walls reinforced by CFRP bars with different shear span ratios: All tested walls in this dissertation have a same shear span ratio of 2.0. However, seismic behavior of walls with different shear span ratios is not clear.
- 2) Compressive behavior of CFRP bars in confined concrete: As described in this dissertation, degradation of the lateral resistance of CFRP bars reinforced concrete walls was triggered by the rupture of CFRP bars under compression. More experimental information about compressive behavior of CFRP bars in confined concrete is desirable to figure out the effect of compressive behavior of CFRP bars on overall seismic behavior of the walls.
- 3) Development of cyclic bond-slip relationship of CFRP bar: The bond-slip relation has a significant effect on seismic behavior of the walls. The use of modified bond-slip model originally proposed by Funato did give a good predication of cyclic curves of CFRP bars reinforcing walls. However, the accuracy of the calculated residual drift by numerical analysis still need to be enhanced, for which modeling of cyclic bond-slip relationship based on more experimental data is inevitable.
- 4) Development of simplified hysteretic model for concrete walls reinforced by CFRP bars: Although a simplified V-R envelop curve model has been proposed in chapter six, a complete simplified hysteretic model including hysteresis loop is also desirable for practical design.
- 5) Fire resistance of concrete walls reinforced by CFRP bars: The fire resistance of

CFRP bars needs to be studied considering that fire may occur after strong earthquakes. One of the methods to enhance the fire resistance of CFRP bars used in shear walls is avoiding spalling of the cover concrete, which can be realized by using such as CFRP sheets to confine the potential plastic hinge region of concrete walls. The effectiveness of this method needs to be experimentally verified in the near future.

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List of Publications

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