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Scientific paper

Performance of Confined Boundary Regions of RC Walls under Cyclic Reversal Loadings

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Abstract

Observed damages in reinforced concrete wall buildings following some recent earthquakes raised concerns about the seismic performance of rectangular RC walls. Damages in RC walls included spalling and crushing of concrete and longitudinal reinforcement buckling at boundaries as well as global buckling. Preliminary studies attributed these damages to the lack of adequate confinement and detailing in wall boundary regions and high axial load level. Prism specimens representing wall boundaries were tested to study the influence of reinforcement detailing, cross-section slenderness, and loading type on the damages, failure modes, and compressive capacity of isolated confined boundary regions of RC rectangular walls. It was found that the tensile strain prior to compressive strain affected the performance of thin wall boundaries and may lead to different failure modes when subjected to cyclic loading. It was also found that dense transverse reinforcement detailing in thin confined boundaries did not improve their compressive capacity. Design and detailing rules to prevent global buckling and reinforcement bar buckling were also evaluated. A Numerical model that takes into account buckling of reinforcement was proposed to simulate response curves of cyclically tested specimens. The model showed the influence of reinforcement buckling behavior on reducing the compressive capacity for elements with buckling of reinforcement failure.

1. Introduction

Reinforced concrete structural walls are commonly used as lateral-load resisting components in medium and high-rise buildings located in earthquake-prone regions. When properly designed and detailed, RC walls are considered to perform well under earthquake loading due to their lateral stiffness and strength. By providing adequate strength and ductility, these walls also behave in a ductile flexural manner. To achieve this goal, lateral instability, fracture or buckling of longitudinal reinforcement should be prevented (Paulay 1986). Observations following the 1985 Chile earthquake (Wood *et al.* 1987) had shown that most RC buildings with wall systems exhibited satisfactory performances. Although detailing requirements for the Chilean wall system as of 1985 were less strict compared to those of the US or Japan. Moreover, RC buildings with a large number of rectangular walls, but without special detailing for the transverse reinforcement, have been reported to perform well during major earthquakes (Wallace and Moehle

1992). A common practice in the design of structural walls in Japan and some other countries is the use of barbell shape cross sections with confined boundary columns that can carry a large amount of axial load. However, modern architecture and design practices promoted the use of slender rectangular walls with the confidence that these planar walls with uniform wall thickness can be designed with adequate ductility.

Following the 2010 Chile earthquake and the 2011 New Zealand earthquake, observed damages in RC wall buildings raised concerns about the seismic performance of rectangular RC walls. In these earthquakes, severe damages happened to RC walls in numerous walled buildings leading to partial or total collapse (Kato *et al.* 2010; Moehle *et al.* 2010). RC Wall damage included spalling and crushing of concrete at boundaries that often spread over the entire wall width, buckled of longitudinal reinforcement under compression and fracture under tension at boundaries (Westenenk *et al.* 2012; Wallace *et al.* 2012). Global wall buckling was also observed in some damaged buildings. **Figure 1** shows typical wall damages following the 2010 Chile earthquake. The damaged wall regions were typically localized over a very limited height of the wall thickness. It was reported that lack of adequate confinement and detailing in boundary regions was one of the main causes of those damages. These observations raised questions about the mechanisms that lead to reinforcing bars buckling, concrete crushing, and global wall buckling, as well as the quantity and configuration of transverse reinforcement at wall boundaries required to ensure good performance by maintaining a stable compressive region and ensuring large deformation capacity without drastic decrease in

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Fig. 1 Typical walls damage in the 2010 Chile earthquake (Wallace *et al.* 2012). Reprinted with permission of the Earthquake Engineering Research Institute.

load carrying capacity. Preliminary studies (Wallace 2012; Talleen *et al.* 2012) indicate that greater amounts of transverse reinforcement may be required for thin walls and that tighter spacing of transverse reinforcement may be required to suppress buckling of vertical reinforcement. In Japan, although the use of walls with boundary columns is still the common practice, the AIJ Standard for Structural Calculations of RC Buildings (AIJ 2010) was revised in 2010 to allow the use of RC walls with rectangular cross-sections.

Literature review has shown that limited studies have been conducted on confined boundary regions, mainly with focus on lateral instability due to out-of-plane buckling. Some researchers revealed that the potential of out-of-plane buckling does not depend only on the compressive strains in wall boundaries, but also on the magnitude of the inelastic tensile strains imposed on wall boundary prior to compressive strain when subjected to reversal cyclic loading (Paulay and Goodsir 1985). Chai and Elayer (1999) conducted an experimental study to examine the out-of-plane stability of RC columns, representing the confined boundary regions of a ductile rectangular RC wall, under large amplitude reversed cyclic tension and compression loading. The study confirmed the critical influence of the maximum tensile strain on the lateral stability of these members. Design recommendations for minimum required wall thickness have been formulated (Paulay and Priestley 1993; Chai and Kunnath 2005). These studies demonstrated the response of ductile boundary elements which resulted in global buckling instead of buckling or fracture of the longitudinal bars. Issues related to buckling of longitudinal reinforcing bars are usually addressed in the design

codes by detailing provisions for the spacing of transverse reinforcement by using ratios of transverse reinforcement spacings (s) to longitudinal bar diameter (d_b) s/d_b . Large s/d_b ratios may result in limited confinement of concrete, and leave longitudinal reinforcement more vulnerable to buckling instability.

Moehle *et al.* (2011) tested several isolated boundary elements, and comparisons were made between elements subjected to compression only and those subjected to 4% tension strain prior to compression, although latter loading type is considered as an extreme situation. This comparison showed different failure modes, and confirmed the vulnerability of wall boundaries to out-of-plane buckling and the reduction of compressive capacity due to large tensile strains prior to compression. Chryanidis and Tegos (2012) tested 5 boundary elements with similar geometry and detailing under different pre-tension strain prior to compression ranging from 0% to 5%. Test results showed that specimens subjected to large pre-tension strains (3% and 5%) showed different failure modes and a significant loss of compressive strength of more than 65%. Massone *et al.* (2014) conducted a test program on 24 specimens of boundary elements with different confinement configuration and slenderness under monotonic compression. Five specimens were subjected to pre-strain in tension of about 2% prior to compression. Test results revealed the effect of pre-tension to reduce compressive capacity by about 15% compared to capacity of specimens tested under monotonic compression. This study clarified the influence of reinforcement detailing, slenderness and loading type (Monotonic and Cyclic) on the compressive capacity, damage progress and failure modes of confined

Table 1 Element specimens cross sections and reinforcement details.

Specimen		Section $b \times l$ (mm)	Layout (Unit: mm)	Long. Reinf	Transv. Reinf.	s/d_b	Loading type
B1-type	1B-M	238×108		10-D10 ($\rho_l = 2.63\%$)	3-D4@80 ($\rho_t = 0.22\%$)	8	Monotonic
	1B-C		Cyclic				
	2B-M				6-D4@80 ($\rho_t = 0.42\%$)	8	Monotonic
	2B-C						Cyclic
	3B-M	242×112			6-D6@80 ($\rho_t = 0.95\%$)	8	Monotonic
	3B-C						Cyclic
	4B-M				6-D6@60 ($\rho_t = 1.27\%$)	6	Monotonic
	4B-C						Cyclic
B2-type	5B-M	238×108		10-D16 ($\rho_l = 7.33\%$)	3-D4@80 ($\rho_t = 0.22\%$)	5	Monotonic
	5B-C						Cyclic
	6B-M	242×112			6-D6@60 ($\rho_t = 1.27\%$)	3.75	Monotonic
	6B-C						Cyclic
C-type	1C-M	363×68		12-D10 ($\rho_l = 3.24\%$)	4-D4@70 ($\rho_t = 0.22\%$)	7	Monotonic
	1C-C						Cyclic
	3C-M	367×72			6-D6@40 ($\rho_t = 1.29\%$)	4	Monotonic
	3C-C						Cyclic

Note: ρ_l is the longitudinal reinforcement ratio $\rho_l = A_l / (b \times l)$, ρ_t is the transverse reinforcement ratio $\rho_t = A_t / (l \times s)$, A_l and A_t are the longitudinal and transverse reinforcement area, respectively.

boundary regions of RC rectangular walls. The study also aimed to assess whether global out-of-plane buckling and buckling of vertical reinforcement could be expected at the boundaries of concrete walls, and to explore the relationship between the phenomena of concrete crushing and reinforcing bar buckling and to determine if current detailing practices are adequate to prevent bar buckling under extreme lateral loading. It is also important to predict the ultimate deformation capacity by building numerical model which takes into account these damage situations.

2. Experimental program

An experimental program was conducted in order to bring insight on the seismic performance of confined end regions of RC rectangular walls. The objective was to investigate the influence of longitudinal and transverse reinforcement detailing, cross-section slenderness and

loading type (Monotonic and cyclic) on their compressive capacity, damage process and failure modes. The behavior of boundary regions in a ductile RC wall subjected to lateral loading was studied by isolating the boundary regions of the wall as axially loaded RC column. Although this approach lacks strain gradient effects expected across the wall section and ignore the contribution of the shear component, the idealization is useful to provide an understanding of the behavior and to identify critical parameters involved during lateral loading of RC walls, where confined boundaries are subjected to large amplitude of tension and compression cycles. It is also important to contribute to make an experimental database related to failures by buckling of reinforcement and global buckling.

2.1 Description of the test specimens

A total of sixteen (16) rectangular elements with two different sectional dimensions (B-type and C-type) hav-

ing approximately similar cross-sectional area were constructed and tested. **Table 1** shows the cross-sectional configurations, the layouts and amounts of reinforcement. The elements were built without cover concrete so that to avoid a sudden drop in the response curves following spalling of cover concrete, since the objective was to assess ultimate behavior and final failure modes. The cross sections dimensions ($b \times l$) for B-type and for C-type specimens represent two levels of slenderness (h/b). The dimensions are measured from the outside of the transverse reinforcement. The shorter side length of the section corresponds to boundary wall thickness. For B-type specimens, specimens from 1B to 4B were constructed with four levels of transverse reinforcement ratio ranging from 0.22% to 1.27%, respectively. Specimens 5B and 6B were constructed with similar transverse reinforcement ratios as for 1B and 4B specimens, respectively, but with larger longitudinal reinforcement ratio. For C-type specimens (1C and 3C), two levels of transverse reinforcement were set and were also similar to transverse reinforcement ratios of 1B and 4B, respectively. For each of these eight configurations, two identical specimens were built to produce sixteen specimens so that each configuration was tested under monotonic compressive load and under cyclic tension and compression reversal load. The last characters in the specimens label stand for loading type, M for monotonic and C for cyclic. D4 (SD295A) deformed reinforcing bars were used for transverse reinforcement for lightly confined specimens and D6 (SD295A) for densely confined specimens. All transverse reinforcement had 135-degree hooks. D10 (SD295A) deformed reinforcing bars were used for longitudinal reinforcement for B1-type ($\rho_l = 2.63\%$) and C-type ($\rho_l = 3.24\%$) specimens, while D16 (SD295A) deformed reinforcing bars were used for B2-type ($\rho_l = 7.33\%$).

Figure 2 shows vertical reinforcement layout of 6B and 1C configurations. Longitudinal reinforcing bars

Table 2 Concrete mechanical properties.

Compressive strength (MPa)	Strain at peak (%)	Young's modulus (GPa)	Splitting strength (MPa)
24.5	0.18	26.3	2.3

Table 3 Reinforcing bars mechanical properties.

Bar	Young's modulus (GPa)	Yield strength (MPa)	Ultimate strength (MPa)
D4	/	363	532
D6	192	365	516
D10	190	347	484
D16	188	325	462
D25	192	381	567

were bent 180-degrees at their ends and hanged to a D25 (SD345) deformed reinforcing bars in the upper and lower stub to ensure good anchorage. D25 bars were also used as longitudinal reinforcement for lower and upper stubs with D10 transverse reinforcement. The tested elements had 600mm height (h) with fixed at both ends to the lower and upper stubs. This height represents the lower portion of the confined boundary in a wall where likely compressive failure may occur. Observations from previous experimental studies indicate that the compressive failure region is quite limited within a height of about 2.5 times the wall thickness (Markeset and Hillerborg 1995; Takahashi *et al.* 2013). The elements were cast vertically in two stages, the lower stub was cast first and then the element and the upper stub as one part with intentionally roughened surface created at lower stub-element interface to insure adherence. **Table 2** and **Table 3** show measured material properties for reinforcing bars and concrete, respectively. A concrete mix with 13mm of maximum aggregate size and 12cm for slump test was used.

Fig. 2 Vertical reinforcement layout of 6B-M/6B-C (left) and 1C-M/1C-C (right).

2.2 Loading Method and Measurement

Figure 3 shows test setup and loading protocol. A Universal Testing Machine with a capacity of 1500kN was used to apply vertical load on the upper stub under the condition of uniaxial tension and compression. Only vertical displacement is possible and the head of the testing machine have no freedom for rotation or lateral displacement. For monotonic tests, the compression load was applied gradually until failure. For cyclic tests axial loading history was determined based on the average strain at the lower part of previously tested RC structural walls (Taleb *et al.* 2014) and previous tests on isolated RC boundary elements (Chai and Elayer, 1999). A ratio of tensile-to-compressive strain of 5 was used as the loading protocol. Thus, the loading cycle consisted of an initial half cycle of axial tensile strain followed by a compression half cycle with a nominal target compressive strain 1/5 of the axial tensile strain. The test was terminated when the resistance of the specimen decreased significantly and the specimen exhibited instability. Thus, two cycles of loading were applied that correspond to yielding tensile strain followed by tensile strains of 0.5%, 1%, 1.5% 2%, 3% and 4%. A load cycle is considered stable if the target compressive strain was reached in two successive cycles without excessive decrease in compressive capacity. The specimens were tested at low rate of monotonic loading, which was in the order of 1mm/min for compression and 4mm/min for tension to insure that no strain rate effects were introduced to damage process. **Figure 4** shows displacement transducers positions for B-type and C-type configurations. Displacement transducers were mounted to both ends of the longer side length of the prism section which was divided into three measuring zones Z1 (Gauges ① and ②), Z2 (Gauges ③ and ④) and Z3 (Gauges ⑤ and ⑥). These zones were set at intervals of 0~50mm for Z1 at the bottom part of the element, 50~550mm for Z2 at the middle, and 550~600mm for Z3 at the top part for B-type elements, and at intervals of 0~40mm (Z1),

40~560mm (Z2) and 560~600mm (Z3) for C-type elements. This difference in insert positions between B-type and C-type was due to transverse reinforcement position. Two displacement transducers (Gauges ⑦ and ⑧) were also installed between upper and lower stub at both sides to check any possible inclination during test. Although variation of configurations and confinement may affects the degree of localization and measured strain, measured strains are compared in an average manner. The nominal axial strain, ϵ_{nom} , was defined experimentally as the strain corresponding to average displacement at both ends of the specimen over its total height h (600mm).

$$\epsilon_{nom} = \frac{1}{2} \left[\frac{(N_1 + N_3 + N_5)}{h} + \frac{(S_2 + S_4 + S_6)}{h} \right] \quad (1)$$

where, N_1 , N_2 and N_3 are displacements corresponding to north side transducers 1, 3 and 5, respectively. S_1 , S_2 and S_3 are displacements corresponding to south side transducers 2, 4 and 6, respectively, and h is the specimen height (600mm).

Fig. 4 Displacement gauges positions.

(a) Test setup

(b) Loading protocol

Fig. 3 Test setup and loading protocol.

3. Experimental Results and Discussion

3.1 Axial load-Axial Nominal Strain Relationships

Figure 5 shows axial load versus nominal axial strain relationships for all specimens. Each plot in the figure represents response relations for both monotonic and cyclic loading. The upper and lower dashed lines indicate, respectively, the calculated loads corresponding to the yielding of longitudinal reinforcements.

$$N_t = A_s f_y \quad (2)$$

and the compressive strength as sum of the concrete uniaxial compressive strength and compressive yield stress of longitudinal reinforcement.

$$N_c = A_s f'_c + A_s f_y \quad (3)$$

where A_c and A_s are the cross-sectional area of concrete gross section and longitudinal reinforcement, respectively, and f'_c and f_y are the concrete compressive cylinder strength and the yield strength of longitudinal reinforcement.

In C-type elements, vibration of concrete during concrete casting was conducted manually using steel rods

Fig. 5 Axial load-Nominal axial strain relationships.

since the use of vibrator was not possible due to the lack of space. Some small honeycombs were observed after removing the formwork. Hence, the unconfined compressive strength was not fully reached. It should be also noted that excessively large compressive strain for 6B-M and 6B-C elements was due to an inclination of the elements prior to extensive crushing. For all specimens, a stable response was observed under low levels of axial tensile strains for element tested under cyclic loading. However, increasing the tensile strain level led to different response. These differences and the comparison monotonic and cyclic loading response are summarized in the following.

It was noted that specimens with thin boundaries (C-type) were not able to fully develop the compressive strength. These configurations could not provide sufficient confinement although the transverse reinforcement ratio was high for 3C configuration. The low confinement ratio and large difference between longitudinal-to-transverse bar diameters led also to a lower compressive capacity. Comparison of load carrying capacity between monotonic and cyclic loadings showed no significant difference for all tested elements.

Failure of 1B, 5B and 1C configurations as well as 2B-C element was due to longitudinal reinforcing bar buckling. 3B, 4B and 6B configurations as well as 2B-M and 3C-M elements failed due to crushing of concrete, while global buckling failure was observed for 3C-C element. Failure mode due to buckling of reinforcement is indicated when apparent longitudinal bar buckling is observed with slight damage in only concrete surrounding longitudinal bars. Failure mode due to concrete crushing is indicated when extensive damage is observed in compressive concrete without any apparent longitudinal bars buckling.

Comparison between monotonic and cyclic response for elements with failure mode governed by buckling of longitudinal reinforcement (1B, 5B, and 1C configurations) showed that prior tensile strain affects considerably the load level at onset of bar buckling. Onset of bar buckling for elements tested under monotonic compression (1B-M, 5B-M and 1C-M) was noted around the peak point, followed by a rapid drop of the load carrying capacity and revealing that their failure was related to longitudinal bar buckling. Following bar buckling, the core concrete could not sustain the total axial load and extensive concrete crushing happened at bar buckling region. Response curves of elements that failed by buckling of longitudinal bars showed a quick decrease of axial load after the peak compressive load was reached. **Figure 6** compares load-strain curve for B-type elements tested under monotonic load. A sudden drop of the capacity was observed and manifested by the rapid concrete crushing at the region of reinforcement buckling.

On the other hand, elements tested under cyclic loading (1B-C, 5B-C and 1C-C) showed onset of bar buckling at lower compressive load before cracks completely closed following unloading from peak tensile strain.

Onset of bar buckling for 1B-C element was observed at approximately -220kN that correspond to about 80% less than the load level of bar buckling under monotonic compression. Onset of bar buckling happened after unloading from the first cycle of 2% tensile strain, similarly to 5B-C element. Onset of bar buckling for 1C-C element started when unloading from the second cycle of 1.5% tensile strain and loading to the corresponding compressive strain, that is compressive strain corresponding to compressive peak load. The following cycle was marked by buckling of several longitudinal bars and capacity drop. This demonstrate vulnerability of slender elements to bar buckling. Response curves of elements which failed due to concrete crushing showed a smoother decrease of load carrying capacity compared to elements with failure mode by longitudinal bar buckling. This smooth decrease was more pronounced as the ratio of transverse reinforcement was higher.

Comparing densely and lightly confined specimens, it was shown that well confined specimens revealed capability to sustain larger tensile strain in a stable manner. However, dense transverse reinforcement detailing added little to the compressive capacity, especially for thin elements. Comparison of compressive capacity of 4B ($\rho_t = 1.27\%$) to 1B ($\rho_t = 0.22\%$) configurations showed an increased capacity of about 16%, while comparison between 3C ($\rho_t = 1.29\%$) and 1C ($\rho_t = 0.22\%$) configurations display similar capacity even though the transverse reinforcement ratio in 3C was set more than 5 times of that in 1C. These observations suggest that it may not be even possible to provide enough confinement in thin sections by close transverse reinforcement spacing because the core concrete width is small and the pattern of concrete crushing indicates that compression strain concentrates over a short height.

Comparing the two levels of slenderness (B-type and C-type), it was shown that although they had similar confined area, the compressive load capacity of C-type elements was in the range of 25% to 40% less than the capacity of B-type elements. This was due to the thin core concrete in C-type elements where a similar confining effect to section with small aspect ratio cannot be obtained and spread of concrete crushing by confined

Fig. 6 Comparison of load- strain curves for B-type elements under monotonic load.

Table 4 Numerical values for observed damage states.

Specimen		Reinf. Comp. Yield.	Peak	Reinf. Buckl.	Global Buckl.	Failure Mode
1B-M	Laod (kN)	-839.3	-1053.2	-1045.3	-	Reinf. Buckling
	Strain (%)	-0.2135	-0.391	-0.4425	-	
1B-C	Laod (kN)	-879.4	-1027.6	-290.2	-	Reinf. Buckling
	Strain (%)	-0.192	-0.378	-0.033	-	
2B-M	Laod (kN)	-607.2	-876	-	-	Concrete Crushing
	Strain (%)	-0.2115	-0.4655	-	-	
2B-C	Laod (kN)	-733.5	-978.8	-327.3	-	Reinf. Buckling
	Strain (%)	-0.1945	-0.4355	-0.0985	-	
3B-M	Laod (kN)	-781.2	-1186.8	-	-	Concrete Crushing
	Strain (%)	-0.199	-0.6185	-	-	
3B-C	Laod (kN)	-685.9	-1095.4	-	-	Concrete Crushing
	Strain (%)	-0.188	-0.6415	-	-	
4B-M	Laod (kN)	-601	-1189.7	-	-	Concrete Crushing
	Strain (%)	-0.18	-0.897	-	-	
4B-C	Laod (kN)	-763.7	-1201.5	-	-	Concrete Crushing
	Strain (%)	-0.205	-0.6295	-	-	
5B-M	Laod (kN)	-834.2	-1045.3	-989.3	-	Reinf. Buckling
	Strain (%)	-0.2005	-0.377	-0.5055	-	
5B-C	Laod (kN)	-894	-1120.8	-477.3	-	Reinf. Buckling
	Strain (%)	-0.201	-0.4045	1.032	-	
6B-M	Laod (kN)	-565.2	-1237.2	-	-	Concrete Crushing
	Strain (%)	-0.2105	-1.5775	-	-	
6B-C	Laod (kN)	-540.8	-1175	-	-	Concrete Crushing
	Strain (%)	-0.176	-1.468	-	-	
1C-M	Laod (kN)	-664.9	-795	-517.7	-	Reinf. Buckling
	Strain (%)	-0.1995	-0.436	-0.628	-	
1C-C	Laod (kN)	-662.1	-730	-256.8	-	Reinf. Buckling
	Strain (%)	-0.1885	-0.2985	-0.226	-	
3C-M	Laod (kN)	-435.7	-719.4	-	-	Concrete Crushing
	Strain (%)	-0.151	-0.618	-	-	
3C-C	Laod (kN)	-532.1	-838.6	-	-140.5	Concrete Crushing
	Strain (%)	-0.1495	-0.5975	-	3.794	

core concrete could not be ensured. Imposing a minimum wall thickness would be an alternative means to suppress failures due to global buckling (Chai and Kunnath 2005) and maintain a stable compression zone.

3.2 Damage process and failure modes

Damage process is presented for each configuration under monotonic and cyclic loading conditions. All elements tested under monotonic compression exhibited a stable behavior without apparent damages until peak load. Following peak load point, different damage evolutions and failures were observed. For cyclically tested elements, horizontal cracks appeared at top and bottom element-stub interface. They also appeared uniformly at transverse reinforcement planes when loading in tension, indicating that these cracks were initiated by the transverse reinforcement. Further tension loading led to widely opened horizontal cracks. **Table 4** gives the numerical values for the observed damage states. Damage evolution and failure modes are described in the following.

3.2.1 B1-type specimens

Figure 7 shows the final damage situation for B1-type specimens. For 1B-M element subjected to monotonic compression, first cracks appeared near corner vertical

reinforcing bars at the top region followed by the spalling of surrounding concrete and buckling of multiple longitudinal bars at this region with a buckling length corresponding to one transverse reinforcement spacing. On the other hand, 1B-C element started damage under compression by spalling of surface concrete at mid-height and the start of buckling of two corner bars with one transverse reinforcement spacing for buckling length. Extensive spalling of concrete was shown in the following compressive cycle with large buckling of longitudinal bars. Buckling length of corner and intermediate supported bars corresponded to one transverse reinforcement spacing while this buckling length for unsupported intermediate bars corresponded to more than two spacing. Crushing of concrete was not so severe at the buckling region following longitudinal reinforcement buckling.

For 2B-M element, damage started with the appearance of multiple vertical cracks at mid-height that quickly led to large spalling of surface concrete followed by crushing of concrete and buckling of several corner and intermediate longitudinal bars over one transverse reinforcement spacing. 2B-C started to damage under compression by the spalling of concrete around some corner bars at the middle and then at the top regions followed by buckling of longitudinal bars at those loca-

Fig. 7 Final damage situation for B1-type specimens.

tions. At further compressive strains, extensive spalling of surface concrete between longitudinal reinforcing bars at mid-height region of the element occurred followed by buckling of many longitudinal bars over one transverse reinforcement spacing. A sudden concrete crushing at that region happened at final stage. A fracture of one longitudinal bar was observed when loading from 3% to 4% tensile strain. Failure modes of 2B configuration was different depending on loading type. 2B-M failed due to crushing of concrete, while failure of 2B-C was attributed to buckling of reinforcement that led to a sudden concrete crushing. Further large strains in tension and compression resulted in an increased number of regions where corner longitudinal bars buckled. This damage situation indicates that the pre-cracks at transverse reinforcement planes due to previous tension strains prior to compressive strain facilitates their buckling compared to element tested under monotonic compression.

3B-M started damage with the appearance of multiple vertical cracks at different locations around corner bars. Further compressive strains led to spalling of surface concrete and crushing of core concrete. For 3B-C, spalling of concrete around a pair of corner bars occurred with core concrete crushing at final stage at top region of the element. Damage evolution for 4B-M and 4B-C elements was similar to 3B-M and 3B-C elements, respectively.

3.2.2 B2-type specimens

Figure 8 shows the final damage situation for B2-type and C-type specimens. For 5B-C, horizontal cracks opened widely and new horizontal cracks formed at mid-spacing between transverse reinforcement at further tensile strains. At final stage, buckling of multiple longitudinal reinforcing bars happened simultaneously at mid-height region over three and four spacing of transverse reinforcement after the spalling of surface concrete between longitudinal reinforcing bars. The transverse reinforcement (D4@80) was not able to contribute effectively in retaining larger longitudinal bars diameter (D16) and preventing them from buckling over large buckling length even though ratios of hoop spacing to longitudinal bar diameter, $s/d_b = 5$, is within the limit of ACI 318-14. This suggests that anti-buckling detailing provisions should also be related to the ratio of longitudinal-to-transverse bar diameters. Buckling of unsupported intermediate bars was more pronounced compared to other bars. 5B-M reached maximum capacity without visible damage, followed by spalling of concrete and buckling of longitudinal reinforcement over two and three transverse reinforcement spacing similarly to 5B-C element.

For 6B-C element, horizontal cracks appeared only at transverse reinforcement planes under tension loading and opened widely as tensile strain increased. At final

Fig. 8 - Final damage situation for B2-type and C-type specimens.

loading stage, both 6B-C and 6B-M failed by crushing of compressive concrete followed by localized buckling of the damaged region, but no buckling of longitudinal reinforcement was observed. The damaged region was located at the lower portion for 6B-M and at the top for 6B-C.

3.2.3 C-type specimens

Both 1C-C and 1C-M specimens failed by buckling of longitudinal reinforcement under compression. Buckling length was observed over two and three transverse reinforcement spacing for 1C-M, while it extended in 1C-C over more than four spacing of transverse reinforcement due to pre-cracks induced by tensile strain. Pre-cracking condition facilitates the buckling of longitudinal reinforcement in addition to the very thin concrete core. Similar to 5B configuration, buckling of unsupported intermediate bars was more pronounced than other bars, suggesting that restraining unsupported intermediate bars in the confined boundary region should be considered, especially for slender walls.

The final failure for 3C-M element was caused due to extensive crushing of compressive concrete at the bottom of element over a very limited height corresponding to approximately two transverse reinforcement spacing. Crushing of concrete for 3C-C was also concentrated at the bottom within limited height, similarly to 3C-M.

However, crushing of concrete in 3C-C was followed by global buckling of the element when unloading from the second cycle of 4% tensile strain indicating that global buckling was driven by prior induced large tensile strain. This phenomena demonstrates the vulnerability of confined boundaries of slender walls to tensile strain excursions prior to compressive strain. Concrete crushing was very limited in height compared to B-type specimens.

In specimens failing due to concrete crushing, concrete crushed over a height ranging approximately between 2 to 3 times element width. Fracture under tension of longitudinal reinforcing bars was not observed excluding one longitudinal bars for 2B-C at tensile strain larger than 3%. Globally, no difference of the failure modes were shown when comparing failures under monotonic and cyclic loading condition. Exception was noted for 2B and 3C configurations. 2B-M failed due to crushing of concrete, but failure of 2B-C was due to buckling of reinforcement that led to a sudden concrete crushing. Also, 3C-M failed due to extensive crushing of concrete, while 3C-C element showed a limited concrete crushing region at the base followed by out-of-plane buckling. Prior crushing assisted the global buckling over almost the total height of the element and resulted in a large out-of-plane displacement.

4. Prediction of failure modes and damage situations

4.1 Potential of out-of-plane buckling

Figure 9 shows the final buckled shape of 3C-C. A vertical line was drawn to highlight the transverse displacement of the buckled element in the figure. Wide cracks, which developed at transverse reinforcement planes as a result of a large yield excursion, did not close prior to full development of maximum compressive strength due to residual tensile strain in the previously yielded longitudinal reinforcement. This damage situation caused a critical condition affecting the lateral stability of the wall (Paulay and Priestley 1993; Chai and Elayer 1999). However, crushing of concrete at the base of 3C-C prior to global buckling contributed in a large out-of-plane displacement when unloading from the second cycle of 4% tensile strain since the base acted as a pin joint. This reveals that both large tensile strain prior to compressive strain and prior crushing affect the global buckling failure mode for slender walls. Imposing a minimum wall thickness would be an alternative means to eliminate global buckling.

Buckling may not be easily perceptible at the design level because their mechanism is difficult to quantify even with the current analysis capabilities. Tendency to buckle in RC walls depend primarily on the wall slenderness ratio and loading history or specifically the maximum tensile strain in the boundary longitudinal reinforcement. Parra and Moehle (2014) suggested that buckling instability might be related to two damage situations. One is that tensile yielding softens the boundary in one direction for subsequent loading in the opposite direction under compression, leading to global lateral instability of an intact wall. The second is that the wall crushes first, leaving an even smaller and irregular cross section, leading to instability of the reduced cross section as a secondary buckling failure. However, failure of 3C-C showed a third damage situation, where prior crushing at the bottom led to a global buckling rather than a local buckling of the crushed region.

Based on buckling theory for prismatic sections under cyclic loading, a relation between the critical slenderness

ratio for the wall boundary element and the maximum tensile strain prior to compressive load, ε_{sm} , was proposed as Eq. (4) (Paulay and Priestley 1993; Chai and Elayed 1999; Parra and Moehle 2014).

$$\frac{b_{cr}}{kh_0} = \frac{1}{\pi} \sqrt{\frac{\varepsilon_{sm} - 0.005}{\beta \xi}} \quad (4)$$

where b_{cr} is the critical wall boundary thickness, h_0 is the clear height, β is the effective depth parameter for longitudinal reinforcement assumed to be 0.8 for two layers of longitudinal bars and 0.5 when a single central layer of bars is used, and ξ is a parameter related to mechanical reinforcement ratio that should satisfy:

$$\xi \leq 0.5 \left(1 + \frac{2m}{0.85} - \sqrt{\left(\frac{2m}{0.85} \right)^2 + \frac{4m}{0.85}} \right) \quad (5)$$

with $m = \rho_f f_y / f'_c$ is the mechanical reinforcing ratio. For practical design, Parra and Moehle (2014) suggest that $\xi = 0.25$. Eq.(4) becomes then:

$$\frac{kh_0}{b_{cr}} = \frac{1}{0.7 \sqrt{\varepsilon_{sm} - 0.005}} \quad (6)$$

Figure 10 compares theoretical relation for wall instability given by Eq. (4) and elements test results. A value of β equal to 1.0 was used in Eq. (4) to consider the total thickness since the specimens were built without cover concrete. The equation may be used to judge the potential of global buckling.

4.2 Potential of longitudinal bars buckling

Buckling of longitudinal reinforcing bars are usually addressed by limiting the ratio of transverse reinforcement spacing to longitudinal bar diameter, s/d_b . Large s/d_b ratios result in limited confinement of concrete, and leave longitudinal reinforcement more vulnerable to buckling instability. ACI 318-14 limits the ratio of s/d_b to 6. Rodriguez *et al.* (1999) introduced a criterion to assess the onset of bar buckling based on monotonic and cyclic tests on isolated reinforcing bars with various s/d_b ratios. Tests indicated that bars subjected to cyclic loading were

Fig. 9 Final buckling shape of 3C-C.

Fig. 10 Comparison with theoretical relation for global buckling instability.

more susceptible to buckling failures than bars subjected to monotonic loading. A strain parameter was introduced as an indicator of the onset of bar buckling. This approach is limited to s/d_b ratio equal to 8 and does not take into account buckling susceptibility over multiple transverse reinforcement spacing.

Based on quasi-static tests on RC columns subjected to lateral loads and constant or varying axial load, Kato *et al.* (1995) proposed a model to estimate the buckling length and the onset of inelastic buckling of corner reinforcing bars. The buckling length is given as a function of the number of transverse reinforcement spacing over where the buckling of longitudinal reinforcement is likely to happen. Transverse reinforcement index, α , was also proposed as a design rule to prevent buckling of longitudinal bars given as:

$$\alpha = \frac{(A_w / A_l)(f_{wy} / f_y)}{s / d_b} \geq 0.039 \quad (7)$$

where A_w and A_l are the areas of transverse and longitudinal reinforcement, respectively, f_{wy} and f_y are yield stress of transverse and longitudinal reinforcement, respectively, and s/d_b is the ratio of transverse reinforcement spacing to longitudinal bar diameter. **Figure 11** shows relation between transverse reinforcement index and observed failure mode for cyclically tested element. Although the lower limit of 0.039 for the index of transverse reinforcement seems to be conservative, the index was able to predict the vulnerable specimens to buckling of longitudinal reinforcement as those with the lower transverse reinforcement ratios. Configuration 5B with the lowest index, showed the most drastic failure due to buckling of longitudinal bars, while buckling of longitudinal reinforcement for configuration 2B was only observed for 2B-C tested under cyclic loading. Among the tested configurations, 2B was considered as a limit between failure due to longitudinal bars buckling and failure by concrete crushing. In 5B configuration, transverse reinforcement did not effectively retain longitudinal bars and prevent them from buckling over large buckling length even though ratios of hoop spacing to longitudinal bar diameter, $s/d_b = 5$, is within the limit of

Fig. 11 Transverse reinforcement index for tested elements.

the ACI 318-14. This suggests that anti-buckling detailing provisions should also be related to the ratio of longitudinal-to-transverse bar diameters. **Figure 12** shows relation between longitudinal-to-transverse reinforcement bar diameters times transverse reinforcement ratio as an index to measure the effectiveness of transverse reinforcement to prevent bar buckling. The index was also able to predict vulnerable specimens to bar buckling. A limit of 1.2 is suggested and this index is considered as complementary to previous rules for eliminating bar buckling.

$$\rho_t d_b / d_l \geq 1.2 \quad (8)$$

with ρ_t is the transverse reinforcement ratio, d_b is the longitudinal reinforcing bar diameter, and d_l is the transverse reinforcing bar diameter.

5. Analytical Prediction of Cyclic Load - Strain Relations

In order to simulate the hysteretic behavior of cyclically tested elements, an accurate and reliable prediction of experimentally observed response is proposed. The model addresses important issues such as the hysteretic behavior in both cyclic compression and tension; the progressive degradation of stiffness of the unloading and reloading curves for increasing values of strain; and the effects of confinement, tension stiffening, and gradual crack closure. The model takes into account concrete damage and hysteresis, while retaining computational efficiency. The monotonic envelope curve of the hysteretic model for concrete in compression follows the monotonic stress-strain relation of modified Kent and Park model (Scott *et al.* 1982) offering a good balance between simplicity and accuracy (**Fig. 13a**). The hysteretic behavior of concrete in both compression and tension were modeled using hysteretic unloading and reloading rules proposed by Yassin (1994) as a set of linear stress-strain relations. The model is able to simulate stiffness degradation for both unloading and reloading. The model provides the flexibility to represent the hysteretic behavior of confined and unconfined concrete in both cyclic compression and tension (**Fig. 13b**).

Fig. 12 Longitudinal to transverse reinforcement index.

Fig. 13 Stress-strain relations for concrete.

Fig. 14 Stress-strain relations for reinforcing bars.

The numerical model used for reinforcing steel was based on Menegotto-Pinto model (Menegotto and Pinto 1973). It was extended by Filippou *et al.* (1983) to include isotropic strain hardening effects as shown in Fig. 14a. To include the effect of buckling of reinforcement, Dhakal and Maekawa (Dhakal and Maekawa 2002) model was implemented. Based on a calibration with experimental data, Dhakal and Maekawa model explicitly provides the compressive stress - strain response of the rebar that is linear for pre-yielding branch and follow Eq. (9) and Eq. (10) for post-yield behavior (Fig. 14b).

$$\frac{\sigma_{sc}}{\sigma_t} = 1 - \left(1 - \frac{f_t}{f_y}\right) \left(\frac{\varepsilon_{sc} - \varepsilon_y}{\varepsilon_i - \varepsilon_y}\right) \quad \text{for } \varepsilon_y < \varepsilon_{sc} \leq \varepsilon_i \quad (9)$$

$$\sigma_{sc} = f_i - 0.02E_s(\varepsilon_{sc} - \varepsilon_i) \geq 0.2f_y \quad \text{for } \varepsilon_{sc} > \varepsilon_i \quad (10)$$

where σ_t and f_{it} are the stresses in the tension envelope corresponding to ε_{sc} (current strain) and ε_i (strain at the intermediate point), respectively. f_y and ε_y are yielding stress and strain, respectively. The coordinates of intermediate point correlated to $\sqrt{f_y} L/d_b$ are given by:

$$\frac{\varepsilon_i}{\varepsilon_y} = 55 - 2.3 \sqrt{\frac{f_y}{100}} \frac{L}{d_b} \geq 7 \quad (11)$$

$$\frac{f_i}{f_{it}} = \alpha \left(1.1 - 0.016 \sqrt{\frac{f_y}{100}} \frac{L}{d_b}\right); \frac{f_i}{f_y} \geq 0.2 \quad (12)$$

Fig. 15 Procedure to determine compressive strength.

where α is a coefficient that takes into account strain hardening which is equal to 0.75 for elastic-perfectly plastic bars, and 1.0 for bars with continues linear hardening. L/d_b is the ratio of the buckling length to the longitudinal bar diameter. Buckling length of reinforcement was evaluated based on Kato *et al.* (1995). Buckling length were evaluated as 3 transverse reinforcement spacing for specimens failing due to buckling

Table 5 Comparison between experimental and simulated peak load and strain.

Specimen	Experiment			Numerical Simulation				Comparison	
	eN_{max} (kN)	$e\epsilon_{@max}$ (%)	Failure mode	eN_{max} (kN)	sN_{max} (kN)	sbN_{max} (kN)	$s\epsilon_{@max}$ (%)	eN_{max} / sbN_{max}	$e\epsilon_{@max}$ / $s\epsilon_{@max}$
1B-M	-1053	-0.391	RB	-912	-957	-957	-0.206	1.10	1.90
1B-C	-1029	-0.378	RB		-946	-815	-0.203	1.26	1.86
2B-M	-876	-0.466	CC		-977	-977	-0.208	0.90	2.24
2B-C	-979	-0.435	RB		-968	-878	-0.208	1.12	2.09
3B-M	-1187	-0.619	CC		-1030	-1030	-0.225	1.15	2.75
3B-C	-1095	-0.642	CC		-1101	-1101	-0.218	0.99	2.94
4B-M	-1195	-0.630	CC		-1062	-1062	-0.236	1.13	2.67
4B-C	-1202	-0.897	CC		-1051	-1051	-0.240	1.14	3.74
5B-M	-1045	-0.249	RB	-1310	-1350	-1350	-0.202	0.77	1.23
5B-C	-1121	-0.278	RB		-1309	-1158	-0.201	0.97	1.38
6B-M	-1237	-1.578	CC		-1398	-1398	-0.228	0.88	/
6B-C	-1175	-1.468	CC		-1392	-1392	-0.281	0.84	/
1C-M	-795	-0.436	RB	-944	-990	-987	-0.206	0.81	2.12
1C-C	-755	-0.299	RB		-990	-789	-0.206	0.96	1.45

Note: eN_{max} and $e\epsilon_{@max}$ are experimental peak load and corresponding strain, respectively. N_c compressive load capacity based on uniaxial concrete strength and longitudinal reinforcement yielding. sN_{max} and sbN_{max} are simulated peak load without and with reinforcement buckling effect, respectively, and $s\epsilon_{@max}$ is the corresponding peak strain. CC: Concrete Crushing, RB: Reinforcement Buckling, GB: Global Buckling.

of reinforcement, and one transverse reinforcement spacing for specimens failing due to concrete crushing of global buckling.

Figure 15 shows the procedure followed to determine the compressive strength. Global buckling vulnerability is judge using Eq. (4), and reinforcement buckling is assessed based on the longitudinal-to-transverse reinforcement index and the ratio of transverse reinforcement spacing to longitudinal bar diameter, buckling model is considered to determine the compressive strength. sN_{max} and sbN_{max} are simulated peak load without and with buckling effect, respectively. **Figures 16 and 17** shows comparison between experimental and analytical axial load - axial strain for specimens tested under cyclic and monotonic loading, respectively. **Table 5** compares compressive strength point and corresponding strain.

The analytical model for cyclically tested specimens captures reasonably well the measured response with hysteretic shape of the response and stiffness degradation. The cyclic properties of the implemented analytical stress-strain relations for steel and concrete produce good correlation for global response, and was also able to predict the compressive strength reduction for specimens failing due to reinforcing bar buckling. For specimens tested under monotonic loading, although compressive strength was overestimated especially for specimens failing due to concrete crushing and global buckling, the model well simulated the post peak branch for specimens failing due to reinforcement buckling.

6. Conclusions

An experimental study was conducted on sixteen RC rectangular columns that idealize confined boundaries of RC rectangular walls to examine the effects of slenderness, reinforcement detailing and loading type on their performance under monotonic and cyclic reversed axial

loading. The following conclusions were drawn.

Three different failure modes were observed depending on confinement and slenderness levels: crushing of compressive concrete, buckling of longitudinal reinforcement, and global buckling of element. Although load carrying capacity between monotonic and cyclic loadings showed no significant difference, loading type may lead to different final failure mode.

Dense transverse reinforcement detailing in thin confined boundaries did not improve the performance of walls. Imposing a minimum wall thickness would be an alternative means to suppress failures due to global buckling of thin walls and efficiently use the confinement. It was also shown that failure due to global buckling is affected by both large tensile strain prior to compressive strain and prior crushing of compressive concrete.

Large transverse reinforcement spacing may result in buckling of longitudinal reinforcement following even limited tensile strain excursions. Intermediate unsupported bars are more susceptible to buckling. Supporting all intermediate bars at the wall confined edge should be considered. Comparison between monotonic and cyclic response for elements with failure mode governed by buckling of longitudinal reinforcement showed that prior tensile strain reduced considerably the load level at the onset of bars buckling. Transverse reinforcement index could be used as a design method of to prevent buckling of longitudinal bar. A design index was also proposed that take into account the case of large longitudinal to transverse reinforcing bar diameters.

The proposed longitudinal-to-transverse reinforcement index along with the ratio of transverse reinforcement spacing to longitudinal bar diameter present a simple but effective anti-buckling measures of reinforcement.

An analytical model that include bar buckling was

Fig. 16 Measured and predicted cyclic axial load - axial strain relations.

proposed to predict cyclic response of tested specimens. The analytical model captures reasonably well the measured response and was also able to predict the compressive strength reduction for specimens failing due to reinforcing bar buckling.

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Fig. 17 Measured and predicted monotonic axial load - axial strain relations.

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