NUMERICAL EVALUATION OF ULTIMATE DRIFT CAPACITY OF SLENDER REINFORCED CONCRETE WALLS

C. Netrattana¹, R. Taleb², H. Watanabe³, S. Kono⁴, D. Mukai⁵, and T. Mukai⁶

ABSTRACT

In recent earthquakes, several reinforced concrete (RC) shear walls were damaged by flexural failures through concrete crushing accompanied with buckling of longitudinal reinforcement in the boundary areas. Slender structural walls have to be designed to ensure that drifts are maintained below the ultimate drift capacity. Thus, the ultimate drift capacity of slender RC shear walls needs to be estimated accurately. In this paper, a parametric study on the seismic behavior of RC shear walls was conducted using a fiber-based model to investigate the influence of basic design parameters including concrete strength, volumetric ratio of transverse reinforcement in the confined area, axial load ratio and boundary column dimensions. This study focused on the ultimate drift capacity for both shear walls with rectangular sections and shear walls with boundary columns. The fiber-based model was calibrated with experimental results of twenty eight tests on shear walls with confinement in the boundary regions. It was found that ultimate drift capacity is most sensitive to axial load ratio; increase of axial load deteriorated the ultimate drift capacity dramatically. Two other secondary factors were: increased concrete strength slightly reduced the ultimate drift capacity while increased shear reinforcement ratio and boundary column width improved the ultimate drift capacity.

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ABSTRACT

In recent earthquakes, several reinforced concrete (RC) shear walls were damaged by flexural failures through concrete crushing accompanied with buckling of longitudinal reinforcement in the boundary areas. Slender structural walls have to be designed to ensure that drifts are maintained below the ultimate drift capacity. Thus, the ultimate drift capacity of slender RC shear walls needs to be estimated accurately. In this paper, a parametric study on the seismic behavior of RC shear walls was conducted using a fiber-based model to investigate the influence of basic design parameters including concrete strength, volumetric ratio of transverse reinforcement in the confined area, axial load ratio and boundary column dimensions. This study focused on the ultimate drift capacity for both shear walls with rectangular sections and shear walls with boundary columns. The fiber-based model was calibrated with experimental results of twenty eight tests on shear walls with confinement in the boundary regions. It was found that ultimate drift capacity is most sensitive to axial load ratio; increase of axial load deteriorated the ultimate drift capacity dramatically. Two other secondary factors were: increased concrete strength slightly reduced the ultimate drift capacity while increased shear reinforcement ratio and boundary column width improved the ultimate drift capacity.

Introduction

Slender reinforced concrete walls are normally used as building’s lateral load resisting systems. The failure modes of RC walls loaded in the in-plane direction includes crushing of concrete in the compression zone, buckling and fracture of longitudinal reinforcement in the boundary column, shear failure of wall panels and boundary columns, shear sliding, and global buckling of boundary columns and wall panels. After 2010 Chile earthquake, a large number of RC walls were severely damaged [1,2]. The observed damage in RC shear walls flexural failure which consisted of spalling

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and crushing of concrete and buckling of vertical reinforcement. Most of concrete crushing propagated over a short height along wall length. This kind of failure led one collapsed RC building, 15-story Alto Rio building [2]. The collapse was caused mainly by flexural failure. The lessons tell us the importance to evaluate properly the flexure ultimate drift capacity. This study focused on flexural failure of slender RC walls. In this failure mode, before bar buckling concrete is damaged when compression strain demand exceeds compression strain limit of concrete [3].

Previous researchers have been conducted experiments to investigate the parameters that affect the ultimate drift capacity of RC walls [4,5,6]. It was found that axial load had a significant influence on flexural strength, ductility and failure mode of structural walls. They also found that a high axial load ratio reduced lateral drift capacity and properly arranged transverse reinforcement increased lateral drift of structural walls.

This paper aims to clarify the effects of five important parameters and the interaction between parameters on the ultimate drift capacity of RC walls with flexure failure: 1. concrete strength, 2. volumetric transverse reinforcement ratio, 3. axial load ratio, 4. boundary column size and 5. boundary column depth. RC walls were simulated by fiber model to assess the ultimate drift capacity caused by crushing of concrete on fracture of longitudinal reinforcement. This study neglects other failure modes; bar buckling before concrete crushing, shear failure, shear sliding and global out-of-plane buckling. Other failure modes will be studied in future.

**Description of Numerical Model**

**Geometric Model**

The reinforced concrete walls are modelled with a fiber based model assuming that plane sections remain plane. In the fiber model, the reinforced concrete cross section of walls is divided into layers stacked along the loading direction. Each steel reinforcing bar is modeled as a single element. Out-of-plane buckling is not considered in the analysis. Effects of confinement are taken into account.

![Figure 1. Elements in the fiber-based model of NC40&NC80 (rectangular section) and BC40&BC80 (barbell shape) [6]](image)

**Material Model**

The stress-strain relationships under cyclic loading for reinforcement and concrete are necessary to calculate stress. The Modified Kent and Park model provides a monotonic envelope curve for concrete in compression that captures the effects of confinement [7]. Unloading and reloading paths follow Karsan and Jirsa’s (1969) model [8] as shown in Fig. 2 (a). Menegotto-Pinto (1973)’s model [9] was used for a nonlinear hysteretic steel model as shown in Fig. 2 (b).
Determining Flexural and Shear Drifts of RC shear walls

The total drift ratio of a cantilever RC shear walls, \( R_t \), is the sum of flexural drift component, \( R_f \), and shear drift component, \( R_s \), as shown in Eq. 1.

\[
R_t = R_f + R_s
\]

(1)

**Flexural Drift component**

Figure 3 (d) presents decomposition of flexural drift component. The flexure drift is assumed to be a combination of elastic deformation and plastic deformation as shown in Eq. 2.

\[
R_f = R_{fe} + R_{fp} = \frac{1}{H}(\Delta_{fe} + \Delta_{fp})
\]

(2)

where \( R_{fe} \) is elastic drift and \( R_{fp} \) is plastic drift ratio, \( H \) is shear span, and \( \Delta_{fe} \) and \( \Delta_{fp} \) are the elastic and plastic drifts, respectively. When a RC shear wall acts as a cantilever, the elastic drift \( \Delta_{fe} \) can be computed using a basic elastic theory as shown in Eq. 3. The plastic curvature, \( \phi_{fp} \), is evaluated by a fiber section analysis assuming the plastic curvature is constant over the plastic hinge length, \( l_p \), the resulting plastic drift, \( \Delta_{fp} \), is given by Eq. 4.

\[
\Delta_{fe} = \frac{QH^3}{3EI}
\]

(3)

\[
\Delta_{fp} = l_p \phi_{fp} \left( H - 0.5l_p \right)
\]

(4)
Figure 3. Flexural Deformation at the Ultimate Condition: (a) Schematic Deformation of a RC Shear Wall under lateral load [10] (b) Bending Moment distribution (c) Actual and Idealized Curvature Distribution (d) Decomposition of Idealized Curvature

Shear Drift component

Beyer et al. [10] proposed an equation to estimate the ratio of shear to flexural deformations for walls controlled by flexure. The equation is given in Eq. 5.

\[
\frac{R_s}{R_f} = 1.5 \frac{\varepsilon_{\text{mean}}}{\phi} \frac{1}{\tan \beta} \frac{1}{H}
\]

(5)

where \( \varepsilon_{\text{mean}} \) and \( \phi \) are the axial strain at the center of the wall section and the curvature, respectively, and derived from moment-curvature analysis. These two variables can be evaluated by a fiber section analysis. The cracking angle \( \beta \) is the cracking angle outside the fan where cracks are approximately parallel as shown in Fig. 3 (a). Beyer et al. [10] suggested that this angle can be assumed to be 45 degrees for simplification.

Ultimate condition for fiber analysis

It is considered that the step has reached the ultimate condition if one of three criteria is met;

1. Load carrying capacity drops 20% of the peak load
2. Extreme concrete fiber in confined area reaches the limit strain of confined concrete, \( \varepsilon_{cu} \)
3. Strain of longitudinal reinforcement in confined area reaches the fracture strain, which was assumed 10%.

The ultimate Condition (2) was described by Pauley and Priestley that limit strain of confined concrete is reached when the transverse steel first fractures. Mander et al. [11] proposed the equation to estimate the ultimate limit strain of confined concrete.

\[
\varepsilon_{cu} = 0.004 + \frac{1.4 \rho_s f_{sh} \varepsilon_m}{f_{cc}^t}
\]

(6)
where \( \rho_s \) is volumetric transverse reinforcement ratio, \( f_{yh} \) is yield strength of the transverse reinforcement, \( \varepsilon_m \) is the steel strain at the maximum tensile stress, and \( f_{cc}^{'} \) is confined compressive strength of the concrete.

**Calibration of model**

The proposed model was calibrated with a series of experimental data. The equivalent plastic hinge length is required to determine the plastic drift. The equivalent plastic hinge length has been studied and several equations have been proposed. Although some researchers studied plastic hinge length in experiments to physically determine the plastic hinge in beams, columns, and walls, the procedure is still controversial. This paper employed the work based on parametric studies by Kono et al. [12] who compared eight equations for the equivalent plastic hinge length and eight \( \varepsilon_m \)s (1%-8%). Based on their fourteen specimens, three combinations of \( l_p \) and \( \varepsilon_m \) that give the best estimate of ultimate drift were found as shown in box in Fig.4. In this paper, the ultimate drift of twenty eight tested specimens were compared to the ultimate drifts calculated by the proposed model using these three sets of \( l_p \) and \( \varepsilon_m \). The test specimens used in this verification process were selected by considering these criteria; 1. experiments over the last 15, 2. flexural failure, 3. symmetry, 4. good end region confinement, 5. no shear sliding effect and 6. \( (\rho_s d_h/d_t) \) index under 4. \( (\rho_s d_h/d_t) \) is the relation between longitudinal to transverse reinforcement bar diameter times shear rebar ratio. This index measures the effectiveness of transverse reinforcement to prevent bucking and called as buckling index. There is a suggestion that if \( (\rho_s d_h/d_t) \) exceeds 1.2, bucking might be prevented. The series of twenty eight specimens were considered mean and standard deviation of ratio of experimental and computed ultimate drift, \( \varepsilon R_u/c R_u \). It can be seen from Fig. 4 that the combination of \( l_p = 0.5 l_w \) and \( \varepsilon_m = 1\% \) gives the mean closest to 1 and lowest standard deviation of \( \varepsilon R_u/c R_u \) among this three sets. Therefore, the parametric study uses half the wall length as the plastic hinge length and 1% as the steel strain at maximum stress. Specimens no. 2, 10-13, 17, 20 and 27 were estimated their ultimate drift capacity in very conservative side by this fiber model.
Table 1. Properties of RC shear wall specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>Reference</th>
<th>Size (mm)</th>
<th>Confined area</th>
<th>Wall panel</th>
<th>$f'_{c}$ (MPa)</th>
<th>Axial Level</th>
<th>Shear span ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>WR-20</td>
<td>Oh et al. 2002</td>
<td>1500x2000</td>
<td>200x200</td>
<td>4-D13</td>
<td>449 D10@200</td>
<td>34.2</td>
<td>0.1</td>
</tr>
<tr>
<td>2</td>
<td>WR-10</td>
<td>Ando cop. 2003</td>
<td>1500x2250</td>
<td>300x150</td>
<td>16-D10</td>
<td>569 D4@45</td>
<td>71.8</td>
<td>0.1</td>
</tr>
<tr>
<td>3</td>
<td>WB</td>
<td>Takenaka et al. 2006</td>
<td>1500x2250</td>
<td>300x150</td>
<td>14-D13</td>
<td>704 D6@65</td>
<td>72.8</td>
<td>0.2</td>
</tr>
<tr>
<td>4</td>
<td>No.1</td>
<td>Oh et al. 2007</td>
<td>1070x1940</td>
<td>268x134</td>
<td>16-D10</td>
<td>433 D4@40</td>
<td>65.8</td>
<td>0.15</td>
</tr>
<tr>
<td>5</td>
<td>No.2</td>
<td>Deng et al. 2008</td>
<td>1000x2000</td>
<td>240x100</td>
<td>4-D12@2-2</td>
<td>6.5 D6@60</td>
<td>74.9</td>
<td>0.15</td>
</tr>
<tr>
<td>6</td>
<td>HPCW-01</td>
<td>Tran and Wallace 2012</td>
<td>2000x4030</td>
<td>260x150</td>
<td>6-D12</td>
<td>601 D6@75</td>
<td>61.3</td>
<td>0.2</td>
</tr>
<tr>
<td>7</td>
<td>HPCW-02</td>
<td>Oh et al. 2007</td>
<td>1000x2000</td>
<td>340x100</td>
<td>4-D12@4-6</td>
<td>6.5 D6@361.6</td>
<td>6.5@100</td>
<td>0.14</td>
</tr>
<tr>
<td>8</td>
<td>HPCW-03</td>
<td>Deng et al. 2009</td>
<td>2500x4030</td>
<td>6-D12@2-2</td>
<td>D5, D4@40</td>
<td>2.24 D8@100</td>
<td>D8@100</td>
<td>0.11</td>
</tr>
<tr>
<td>9</td>
<td>I-1</td>
<td>Murakami et al. 2009</td>
<td>1120x2140</td>
<td>210X134</td>
<td>16-D10</td>
<td>409 D4@40</td>
<td>45.6</td>
<td>0.11</td>
</tr>
<tr>
<td>10</td>
<td>SW6-1</td>
<td>Zhang et al. 2010</td>
<td>1000x2000</td>
<td>200x125</td>
<td>6-D10</td>
<td>352 D4@80</td>
<td>65.2</td>
<td>0.15</td>
</tr>
<tr>
<td>11</td>
<td>SW6-2</td>
<td>Murakami et al. 2009</td>
<td>1120x2140</td>
<td>210X134</td>
<td>16-D10</td>
<td>409 D4@40</td>
<td>65.2</td>
<td>0.15</td>
</tr>
<tr>
<td>12</td>
<td>SW6-3</td>
<td>Zhang et al. 2010</td>
<td>1000x2000</td>
<td>200x125</td>
<td>6-D10</td>
<td>352 D4@80</td>
<td>65.2</td>
<td>0.15</td>
</tr>
<tr>
<td>13</td>
<td>SW6-4</td>
<td>Murakami et al. 2009</td>
<td>1120x2140</td>
<td>210X134</td>
<td>16-D10</td>
<td>409 D4@40</td>
<td>65.2</td>
<td>0.15</td>
</tr>
<tr>
<td>14</td>
<td>SW6-5</td>
<td>Zhang et al. 2010</td>
<td>1000x2000</td>
<td>200x125</td>
<td>6-D10</td>
<td>352 D4@80</td>
<td>65.2</td>
<td>0.15</td>
</tr>
<tr>
<td>15</td>
<td>SW6-6</td>
<td>Murakami et al. 2009</td>
<td>1120x2140</td>
<td>210X134</td>
<td>16-D10</td>
<td>409 D4@40</td>
<td>65.2</td>
<td>0.15</td>
</tr>
<tr>
<td>16</td>
<td>SW6-7</td>
<td>Zhang et al. 2010</td>
<td>1000x2000</td>
<td>200x125</td>
<td>6-D10</td>
<td>352 D4@80</td>
<td>65.2</td>
<td>0.15</td>
</tr>
<tr>
<td>17</td>
<td>SW6-8</td>
<td>Murakami et al. 2009</td>
<td>1120x2140</td>
<td>210X134</td>
<td>16-D10</td>
<td>409 D4@40</td>
<td>65.2</td>
<td>0.15</td>
</tr>
<tr>
<td>18</td>
<td>SW6-9</td>
<td>Zhang et al. 2010</td>
<td>1000x2000</td>
<td>200x125</td>
<td>6-D10</td>
<td>352 D4@80</td>
<td>65.2</td>
<td>0.15</td>
</tr>
<tr>
<td>19</td>
<td>SW6-10</td>
<td>Murakami et al. 2009</td>
<td>1120x2140</td>
<td>210X134</td>
<td>16-D10</td>
<td>409 D4@40</td>
<td>65.2</td>
<td>0.15</td>
</tr>
<tr>
<td>20</td>
<td>SW6-11</td>
<td>Zhang et al. 2010</td>
<td>1000x2000</td>
<td>200x125</td>
<td>6-D10</td>
<td>352 D4@80</td>
<td>65.2</td>
<td>0.15</td>
</tr>
</tbody>
</table>

* $l_w \times h$ are external wall length and wall height, respectively. $\rho_s$ is shear rebar ratio in confined area. $t_{w}$ is wall thickness.

Figure 4. Comparison of experimental and computed ultimate drift from fiber analysis
**Parametric Study**

A parametric study was conducted to investigate the influence of important factors in design on the seismic behavior of RC shear walls, particularly ultimate drift capacity. Five variables were studied: concrete strength, $f_c'$, volumetric transverse reinforcement ratio in the confined area, $\rho_s$, axial load ratio, $\eta$, boundary column width, $B$ and boundary column depth, $D$. Axial load ratio is defined as ratio of axial load to compressive concrete cross section capacity, $N/(A_c \times f_c')$. $A_c$ is wall cross section area. Rectangular cantilever structural walls (rectangular shape) and cantilever structural walls with columns at the end regions (barbell-shape) were analysed in various configurations. To isolate the effects of the parameters in this study, it was decided to keep the moment capacity as constant as possible. Therefore, tension force from longitudinal reinforcement was held constant by keeping the same amount of longitudinal reinforcement (568 mm$^2$ is equivalent to 8-D10) while longitudinal reinforcement was lumped to 125 mm from extreme tensile fibers to have the steel centroid at the same location in every case. Variables affect the ultimate drift were set at prototype case, Table 2. The dimension and reinforcement details for prototype case are shown in Fig. 5. When the influence of one variable was studied, all other variables were fixed at the prototype case values. Plastic hinge length and limit strain of transverse reinforcement used results from calibration ($l_p = 0.5l_w$ and $\varepsilon_m = 1\%$).

![Figure 5. Dimension and reinforcement details of prototype specimens](image)

**Table 2. Prototype case**

<table>
<thead>
<tr>
<th>$f_c'$ (MPa)</th>
<th>30</th>
<th>Wall height (mm)</th>
<th>3,000</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho_s$ (%)</td>
<td>0.8 (D6@70)</td>
<td>Wall length (mm)</td>
<td>1,750</td>
</tr>
<tr>
<td>$\eta^*$</td>
<td>0.1</td>
<td>Long. rebar in confined area</td>
<td>8-D10 (0.91%)</td>
</tr>
<tr>
<td>$B \times D$ (mm)</td>
<td>250x250</td>
<td>Gross section area (mm$^2$)</td>
<td>225,000</td>
</tr>
</tbody>
</table>

$^* N/(A_c \times f_c')$, $N$ axial load, $A_c$ Wall cross section area
Results

Influences of Concrete Strength

As can be seen from Fig. 6, increasing concrete strength from 20MPa to 80MPa slightly decreases ultimate drift when \( \eta \geq 0.075 \). The ultimate drift capacity decreases from 4.75% to 4.0% for walls with boundary column, \( \rho_s = 0.8\% \), \( \eta = 0.1 \), \( B = 250\) mm and \( D = 250\) mm. In contrast, higher concrete strength slightly increases the ultimate drift capacity when low axial load were applied, \( \eta \leq 0.05 \). Increasing concrete strength from 20MPa to 80MPa increases the ultimate drift from 5.75% to 6.15% for wall with boundary column, \( \rho_s = 0.8\% \), \( \eta = 0.05 \), \( B = 250\) mm and \( D = 250\) mm.

Influences of Transverse Reinforcement Ratio

As can be seen from Fig. 7, increasing transverse reinforcement in confined area improves the ultimate drift capacity. Increasing transverse reinforcement from 0.2% to 1.2% increases the ultimate drift capacity from 3.29% to 5.02% for wall with boundary column, \( f'_c = 30\) MPa, \( \eta = 0.1 \), \( B = 250\) mm and \( D = 250\) mm. These trends are true for \( \eta \geq 0.05 \) except when \( \eta = 0 \) and \( \rho_s \geq 0.8\% \), the trend is constant since the failure is controlled by Condition (3) as shown in dash circle in Fig.7.

Influences of Axial Load Ratio

As can be seen from Fig. 8, increasing of axil load from 0.0 to 0.4 decreases the ultimate drift capacity considerably, from 7.00% to 0.57% for walls with boundary column, \( f'_c = 30\) MPa, \( \rho_s = 0.8\% \), \( B = 250\) mm and \( D = 250\) mm. The trends are same for different level of \( \rho_s \).

Influences of Boundary Column

Figure 9 shows that lager boundary column width increases the ultimate drift. At the same boundary column width, \( B \), some RC walls have the ultimate drift lower than others because their neutral axis stay outside of boundary column as presented in dashed circles. For 200 mm of column depth, the ultimate drift increases from 2.40% to 5.95% since column width enlarges from 128.6 to 350 mm. If neutral axis locates inside boundary columns, the neutral axis depth remain constant regardless of boundary column depth, \( D \). In this case, the ultimate drift is constant even if boundary column depth changes.
Figure 6  Influences of concrete strength on ultimate drift for different axial load ratio

Figure 7  Influences of transverse reinforcement ratio on ultimate drift for different axial load ratio

Figure 8  Influences of axial load ratio on ultimate drift for different transverse reinforcement ratio

Figure 9  Influences of boundary column dimensions on ultimate drift transverse reinforcement ratio

Conclusions

The effects of five parameters including concrete strength, volumetric ratio of transverse reinforcement in the confined area, axial load ratio and boundary column size on the ultimate drift of slender RC walls were numerically studied with a calibrated fiber model. Five parameters were varied in these ranges: concrete strength from 201 MPa to 80 MPa, transverse reinforcement ratio from 0.2 % to 1.2 %, axial load ratio from 0.0 to 0.4, boundary column width from 128.6 m to 350 mm and boundary column depth from 100 mm to 450 mm. The results can be summarized as below.

- The parametric study showed that axial load was the most influential parameter. Increasing of axial load from 0.0 to 0.4 resulted in ultimate drift capacity deteriorating 6.2% in average. Different axial load level easily changed trends of the ultimate drift capacity due to other four parameters.
- Increasing transverse reinforcement in confined area improved the ultimate drift capacity. Increasing transverse reinforcement from 0.2% to 1.2% increased the ultimate drift capacity by 1% in average.
- Increasing concrete strength from 20MPa to 80 MPa slightly reduced the ultimate drift capacity by 0.6% in average for moderate to high axial load ratio ($\eta \geq 0.075$). In contrast,
increasing concrete strength slightly increased the ultimate drift capacity by 0.5% for low axial load \((\eta \leq 0.05)\).

- Larger boundary column width increased the ultimate drift capacity. Increasing of boundary column width from 128.6 mm (rectangular section) to 350 mm (wall with boundary column) increased the ultimate drift capacity 3.6% for boundary column depth of 200 mm.

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