NUMERICAL STUDY ON THE ULTIMATE DEFORMATION OF RC STRUCTURAL WALLS WITH CONFINED BOUNDARY REGIONS

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ABSTRACT

For accurate assessment of performance levels in reinforced concrete (RC) members, it is important to well define deformation limits at particular damage states. For RC walled building, investigation of the deformation limits of RC structural walls is required to define limit states and corresponding limiting values. Numerical investigations were carried out on barbell shape and rectangular RC walls with confined boundaries to evaluate response curves and ultimate deformations. A nonlinear 2D and 3D finite elements (FE) models were built in order to simulate the load-deformation relations under monotonic loading as well as cracking and damage patterns of previously tested walls. The FE models were able to simulate the backbone curves with good accuracy as well as the ability of boundary columns in reducing damage level. The 3D FE model simulated very well the ultimate deformation compared to 2D models. A sectional fibre model combined with plastic hinge length and shear deformation component is proposed in order to simulate the backbone curves and the ultimate deformation with less computational cost compared to 3D FE analysis. The model was able to provide relatively accurate backbone curves with very good estimation of ultimate drift.

Keywords: Structural RC walls; confined boundary region; ultimate drift; finite element analysis; sectional analysis

1. INTRODUCTION

Reinforced concrete structural walls are frequently used as the primary component of the lateral load-resisting system in buildings located in earthquake prone regions because of their substantial contribution to building lateral stiffness and strength. When properly designed, these structural walls can also behave as ductile flexural members. To achieve this goal, designers should provide adequate strength and displacement capacity. Hence, several experimental and analytical studies were conducted to investigate the behavior of RC structural walls under lateral loads in order for designers to predict their structural performance when subjected to severe seismic excitations. A proper simulation of the ultimate deformation capacity of RC structural walls leads also to a rational determination of the behavior factor or response modification factor used in seismic design codes. A research program was undertaken in order to study the effects of end regions confinement on the seismic performance of moderate aspect ratio type of structural walls (Taleb et al., 2014, 2016). Four 40%-scale RC walls having different cross sectional configurations and transverse reinforcement at their confined end regions of were constructed and tested under lateral cyclic reversed loading. The test specimens included two specimens with boundary columns and two other specimens with rectangular shape section. It was shown that damaged regions due to concrete crushing in rectangular walls spread widely over the lower portion of the walls. The damage tended to spread horizontally

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towards wall center and was limited in height. It was also shown that boundary columns in barbell shape walls could effectively enhance the wall performance by increasing its ultimate deformation capacity and reducing damage level in the wall panels.

2. REVIEW OF EXPERIMENTAL PROGRAM

2.1 Wall Specimens and test procedure

Experimental study were conducted on four 40% scale structural walls designed and constructed by changing the configuration of section (barbell-shape and rectangular sections) and the amount of shear reinforcement in confined regions (Taleb et al., 2014). The wall specimens were tested under reversal quasi-static cyclic loading with displacement control until collapse. A total axial force of 1500kN was applied constantly by two hydraulic jacks to keep the axial load level of 0.20 for confined region, corresponding to 0.11 for the total area of the section. Specimens BC40 and BC80 had confined boundary columns and NC40 and NC80 had no boundary columns but confined boundary regions instead with same thickness as for the wall panel (Figure 1). The four specimens had same width (1750mm), nearly same total section area (2250cm$^2$ for BC40 and BC 80 and 2240cm$^2$ for NC40 and NC80) as well as confined boundary region area (625cm$^2$ for BC40 and BC 80 and 666cm$^2$ for NC40 and NC80). Table 1 and Table 2 give wall specimens and materials properties, respectively. Wall specimens were classified as intermediate aspect ratio walls and designed to fail in flexure, with shear safety factor defined as the ratio of shear capacity to flexural capacity more than 1.5.

Figure 1. Reinforcement detailing of the wall specimens
### Table 1. Properties of the wall specimens

| Specimen       | $h_w/l_w$ (mm) | $a_s$ (Shear span) | Confined end regions | Wall panel | $\mu$ | $c_{su}$ (kN) | $c_{su}$ (kN) |
|----------------|----------------|---------------------|----------------------|------------|-------|---------------|---------------|
|                |                |                     | $A_{ef}$ (mm$^2$)   | $p_{tf}$ | $t_w$ (mm) | $p_{wh}$ = $N/A_{ef}f'_c$ | $c_{su}$ (kN) | $c_{su}$ (kN) |
| BC40/BC80      | 1.6            | 1.71                | 250×250              | 0.91       | 80    | 0.40          | 0.13          | 532            | 761            |
| (2800/1750)    | (3000mm)       |                     | 128×520              | 1.29       | 128   | 0.25          | 0.11          | 586            | 867            |
| NC40/NC80      |                |                     |                      |            |       |               |               |                |                |
|                |                |                     |                      |            |       |               |               |                |                |

### Table 2. Measured mechanical properties of concrete and reinforcement

| Specimen       | Concrete                  | Reinforcement         |                |                |                |                |                |
|----------------|---------------------------|------------------------|----------------|----------------|----------------|----------------|----------------|
|                | Compressive strength (MPa)| Young's modulus (GPa)  | Splitting strength (MPa) | Reinf. bars | Yield strength (MPa) | Young’s modulus (GPa) | Tensile strength (MPa) |
| BC80/BC40      | 59.5                      | 30.9                   | 5.10           | D6            | 387            | 189            | 496            |
| NC80/NC40      | 52.5                      | 30.1                   | 3.66           | D10           | 377            | 194            | 533            |

### 2.2 Observed Damages and failure modes

All specimens behaved in a flexural manner by yielding of the longitudinal reinforcement, reached the peak point and deformed until failure without significant degradation of lateral load carrying capacity. At 0.05% drift ratio, flexural cracks started to appear in the lower part of the tensile region. The number of flexural cracks increased along the confined regions height and progressed into flexural-shear cracks at drift ratio of 0.5% with the yielding of tensile longitudinal reinforcement. Crack openings in NC40/NC80 specimens were larger than those in BC40/BC80 specimens. Figure 2 shows crack patterns at peak load for all specimens.
The longitudinal reinforcement in confined end regions yielded during the cycle of $R=\pm 0.1\%$ or $\pm 0.25\%$ for both walls configuration, although the yielding for NC’s specimens tended to happen earlier than for BC’s specimens. BC40 and BC80 showed no degradation of load carrying capacity until the failure while NC40 and NC80 showed some degradation after reaching peak load due to crushing of core concrete that quickly followed the peak point. It was remarkable that BC40 and BC80 could further sustain load capacity for a larger interval of deformability compared to NC40 and NC80. The ultimate failure was caused by crushing of confined concrete and buckling of longitudinal reinforcement in the compression zone. The performance of wall with boundary columns was better than that of rectangular walls with similar shear span to wall length ratio in terms of drift capacity. Boundary columns also showed the ability to reduce damage level in wall panel since they carry a large amount of axial force that reduces axial stress level in wall panels.

3. NONLINEAR FINITE ELEMENTS ANALYSIS

3.1 2D Finite Element Analysis

Numerical analyses were conducted under monotonic loading to investigate the envelope of lateral load response of the tested walls as well as the damage distribution. The 2D nonlinear analysis was conducted using FE program FINAL (ITOCHU, 2011). Figure 3 shows FE mesh for BC’s specimens. Four-node plane stress quadrilateral elements were used to model the RC walls. The foundation and loading beams were assumed to behave elastically. All nodes at the bottom of the foundation beam were pin-supported to restrain vertical and lateral displacement. The constant axial loads on the top of boundary regions were applied in the first step and then the lateral load was applied at the loading beam center point under displacement control.

Truss elements were used to model the vertical reinforcements in confined boundary regions considering bond effect, which was modelled using Elmorsi model (Elmorsi et al., 2000). Stress-strain relation for reinforcement material follows Ciampi’s model (Ciampi et al., 1982). All Horizontal and vertical reinforcements in wall panels as well as transverse reinforcement in confined regions were smeared assuming a perfect bond. The modified Ahmad model (Naganuma, 1995) for the compressive stress-strain relation of concrete was used for both ascending and descending branches for confined and unconfined concrete. Mechanical properties of material used in the analysis are thus given in Table 2. The Kupfer-Gerstle’s failure criterion was adopted for failure in biaxial compression and in tension-compression. The Naganuma model was adopted for concrete tension stiffing (Naganuma et al., 2004). Uniaxial tensile strength is used for judging cracks under uniaxial and biaxial tension.
Stress-strain relationship is assumed linear up to cracking. The smeared crack model with a fixed angle concept was used to express cracking of concrete. The shear transfer model after cracking proposed by Naganuma was used (Naganuma et al., 2004).

Figure 4 shows cyclic lateral load-drift angle relationships obtained experimentally and monotonic envelop obtained by 2D FE analysis. The ultimate drift was defined by either 20% degradation of load carrying capacity from the peak load or the maximum observed behavior of drift. The results show that the model is capable of simulating the entire steps of the nonlinear behavior of the concrete wall such as initial stiffness, cracking, steel yielding, and peak load with good accuracy.

Table 3 shows comparison of ultimate drift point between experimental and 2D FE analysis. Although the model tends to underestimate the ultimate deformation points, the model well captures their trend since ultimate drift of BC's specimens are larger than those of NC's specimens, and that for the same wall configuration, ultimate drift in specimens with 40mm transverse reinforcement spacing is larger than those with 80mm spacing.

Figure 5 illustrates cracks distribution and damage pattern at ultimate. Crack distribution is less spread in the case of walls with boundary elements compared to that of rectangular walls. Damage for walls with boundary column is concentrated at the outside bottom of boundary columns, while for walls without boundary damage extended along the bottom of confined regions. This is because boundary columns carry a large amount of axial force to reduce axial stress level of wall panels to reduce their damage. The built model predicted damage pattern quite well, and has predicted the ability of boundary columns in reducing damage level and crack distribution.
3.3 3D Finite Element Analysis

Numerical analyses with 3D FE model were also conducted under monotonic loading to verify the ability of 3D modelling for the estimation of ultimate drift, since 2D model was not able to well capture it. In 3D FE model, eight-node elements were used to model the RC walls. The foundation and loading beams were assumed to behave elastically. Similar constitutive material models used for 2D analysis were also used for 3D analysis. All reinforcements, including longitudinal reinforcing bars in confined regions were smeared assuming a perfect bond with concrete. The analysis employed Ottosen’s four-parameter model to define the failure criterion of concrete. Figure 6 shows cyclic lateral load-drift angle relationships obtained experimentally and monotonic envelop obtained by 3D FE analysis. The analysis tends to simulate slightly higher initial stiffness. However, the analytical backbone curve agrees very well with the experimental one until ultimate drift point, especially in positive loading direction. Similarly to 2D models, 3D models could predicted the ability of boundary columns in reducing damage level and crack distribution, since crack distribution is less spread in the case of walls with boundary elements compared to that of rectangular walls (Figure 7). Table 3 shows comparison of ultimate deformation point between experimental and 3D FE analysis. The model estimates very well the ultimate deformations of the tested wall specimens.
4. FIBRE SECTIONAL ANALYSIS

A sectional fiber model analysis was conducted to compute the backbone lateral load-drift angle relations as well as to estimate the ultimate lateral drift based on the plastic hinge length and moment-curvature analysis (Figure 8). The wall section was divided into small concrete elements along the width direction and each longitudinal reinforcing bar was modelled as an independent steel element (Figure 9(a)). The monotonic envelope curve for plain and confined concrete in compression follows the modified Kent and Park model (Scott et al., 1982). The tensile contribution of concrete was neglected. The numerical model used for reinforcing steel was based on Menegotto-Pinto model as extended by Filippou et al., (1983) to include isotropic strain hardening effects.

The total drift is obtained by the sum of the flexural component and the shear component. The flexural component is computed by Equation 1 as the sum of the elastic and the plastic components based on the curvature distribution. The curvature is divided into elastic and plastic curvatures, and each curvature was used to derive elastic drift, $\delta_{fe}$, and plastic drift, $\delta_{fp}$, as Equation 2 and Equation 3.
\[ R_f(\%) = \frac{\delta_{pf} + \delta_{fp}}{H} \times 100 \]  

with,

\[ \delta_{pf} = \frac{QH^3}{3EI} \]  

\[ \delta_{fp} = \frac{1}{2} \phi_p l_p^2 + \phi_p l_p (H - l_p) \]  

Where \( Q \) is the lateral load, \( H \) the wall height (3000mm), \( E \) Young’s modulus of concrete, \( I \) the second moment of inertia of the wall section, \( \phi_p \) the plastic curvature, \( l_p \) the plastic hinge length. The plastic hinge length corresponds to the yielding of longitudinal reinforcement and plastic curvature distribution. The plastic hinge length calculations significantly influence the estimation of the force-displacement response of that wall in the inelastic region. Existing plastic hinge length equations are usually proposed for RC columns and applicable for RC walls. Observations from the tested walls have shown that the damage region was limited in height and tends to spread more horizontally toward wall center. Similarly, 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). Hence, the plastic hinge length was estimated to be 2.5 times the wall panel thickness.

The shear deformation of walls is estimated using the empirical equation developed by Beyer et al. (2011) as given by Equation 4. This empirical equation was developed based on a series of experimental and analytical studies of slender reinforced concrete walls under seismic loading. The shear deformation component was added to the flexural component to obtain the total deformation without considering flexure - shear interaction.

\[ \delta_s = 1.5 \delta_f \left( \frac{\varepsilon_{m}}{\phi \tan \beta} \right) \frac{1}{H_w} \]  

With

\[ \tan \beta = \frac{j_d}{V} \left( f_c t_w + \frac{A_{sw} f_{yw}}{s} \right) \]  

\( \beta \leq 90^\circ \)  

where, \( \delta_s \) is the flexural lateral displacement, \( \beta \) is the crack angle, \( \varepsilon_m \) is the axial strain at the center of the wall section, \( \phi \) is the curvature of the wall section, \( j_d \) is the lever arm between compression and tensile resultants, \( V \) is the shear force, \( f_c \) is the tensile strength orthogonal to the crack, \( t_w \) is the wall thickness, \( A_{sw} \) is the area of the shear reinforcement, \( f_{yw} \) is the yield strength of shear reinforcement, and \( s \) is the spacing of shear reinforcement.

The ultimate drift was computed based on the limit compressive strain, \( \varepsilon_{cu} \), proposed by Mander et al. (1988).

\[ \varepsilon_{cu} = 0.004 + \frac{1.4 \rho_s f_{sh} \varepsilon_{im}}{f'_c} \]  

where \( \rho_s \) is the volumetric ratio of transverse reinforcement in confined end regions, \( f_{sh} \) the yield strength of confining reinforcement, \( \varepsilon_{im} \) the fracture strain of confining reinforcement and 0.005 was used based on reinforcing bars material test, \( f'_c \) the compressive strength of confined concrete. Figure 9(b) shows stress-strain relations for confined concrete regions of the tested wall along with limit compressive strain, \( \varepsilon_{cu} \), computed by Equation 6 represented in the figure by red diamond. In the analysis, when the extreme compressive concrete fiber reached the limit compressive strain, \( \varepsilon_{cu} \), the analysis was terminated and the corresponding drift was considered the ultimate drift.
The computed relations between lateral load, $Q$, and lateral drift angle are compared with the experimental hysteresis curves in Figure 10. Although the computed peak load is slightly smaller than the experimental value, the computed backbone curve well simulates envelop of experimental results. It is noted that the flexural ultimate drift is especially well simulated (Table 3) with less computational effort compared to 3D FE analysis. For the sake of ultimate drift, only envelope numerical results are presented but the fiber model is able to do hysteretic analysis based on hysteretic material models.

Figure 9. (a) Walls sectional fiber meshing and (b) stress-strain relations for concrete with limit compressive strains

Figure 10. Experimental hysteretic (Taleb et al., 2014) and fiber sectional analysis lateral load - drift angle relations
### Table 3. Comparison between experiment and analysis for ultimate drift point

| Specimen | Experiment $R_{exp}$ (%) (+)/(-) | 2D FEM $R_{ana}$ (%) | Fiber $R_{ana}$ (%) | 3D FEM $R_{exp}/R_{ana}$ (%) |
|----------|----------------------------------|----------------------|---------------------|-----------------------------|
| BC40     | 4.00/-2.75                       | 2.32                 | 3.69                | 2.32                        |
| BC80     | 2.00/-2.00                       | 1.72                 | 1.97                | 1.02                        |
| NC40     | 2.38/-2.00                       | 1.32                 | 2.22                | 1.09                        |
| NC80     | 1.50/-1.50                       | 1.07                 | 1.43                | 1.58                        |

Note: (+)/(-) correspond to positive and negative loading directions, respectively. The ratio of experimental and analytical lateral drift was calculated based on the average value of the experimental ultimate drift between positive and negative loading directions.

### 5. CONCLUSIONS

Different numerical methods were used and summarized in this paper for simulating backbone curves and ultimate deformation capacity of reinforced concrete walls with confined boundaries. All used analysis methods have well showed experimental observations regarding boundary columns that can effectively enhance the wall performance by increasing its ultimate deformation capacity and reducing damage level in the wall panel. The built 2D and 3D FE models predicted damage pattern quite well, and has predicted the ability of boundary columns in reducing damage level and crack distribution since boundary columns carry a large amount of axial force, which reduce axial stress level in wall panels. In this manner, boundary columns can contribute effectively in preventing failure mode due to wall buckling, especially when subjected to high axial load. The built 2D and 3D FE models were able to simulate the entire steps of the nonlinear behavior of the concrete wall such as elastic region, cracking, steel yielding and peak load with relatively good accuracy. 3D model could simulate the ultimate deformation points with very good accuracy; however, the 3D nonlinear FE models are time consuming. The fiber model based on the plastic hinge length and moment-curvature analysis easy and interesting alternative for simulating the envelop response curve for RC walls with confined boundaries. In this manner, the Mander’s limit compressive strain is a good measure for the ultimate drift.

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