FULL-SCALE EXPERIMENT ON NON-STRUCTURAL R/C WALLS FOCUSED ON FAILURE MODES AND DAMAGE MITIGATION

Masanori TANI1, Tomohisa MUKAI2, Masaya OGURA3, Rafik TALEB4 and Susumu KONO5

ABSTRACT

In the 2011 Tohoku Earthquake, many R/C non-structural walls were heavily damaged and the function continuity was broken in many buildings. In this research, analytical and experimental study was conducted to obtain fundamental information of R/C non-structural wall, such as failure mode and relation between lateral deformation and damage state. Failure mode of the non-structural walls which suffered severe damage in the 2011 Tohoku Earthquake was discussed based on the capacity estimation. Four full-scale R/C mullion wall specimens were constructed on the basis of the case study results and tested. The results of case study implied that considerable axial compression load was applied on the walls. Also in the loading test, the specimen with axial compression load showed similar failure mode as seen in the actual buildings. Improvement of maximum lateral capacity and deformation capacity was observed by increasing the amount of horizontal web reinforcement and installing of hook anchorage. In addition to discussions on maximum lateral capacity and failure mode, fundamental experimental data about damage condition especially after unloading was also shown.

INTRODUCTION

In the 2011 Tohoku Earthquake, structural damage to buildings was not particularly heavy in comparison with the observed JMA seismic intensities, although a part of R/C buildings which was designed by previous building code was heavily damaged. On the contrary, non-structural R/C walls in many government buildings and residential buildings were heavily damaged as shown in Photo.1. In Japan, R/C wall member with large opening which cannot be treated as structural wall by regulations is called “non-structural wall” and its capacity is not generally considered in the structural calculation. Damage of such member does not affect to the safety of building, but the continuity and rehabilitation of building function. Some buildings whose non-structural R/C walls were heavily damaged were given up their rehabilitation and demolished due to the problems of huge repair cost etc., even if the structural damage was not so significant. Behind this situation, there is a problem that structural designers cannot predict what will happen on non-structural wall because information about the relations between structural frame deformation and non-structural wall damage (ex. residual crack width and length, concrete spalling area, etc.), and the effects of damage to the function continuity and reparability (i.e. repair cost) of buildings are extremely limited.

1 Research Engineer, Building Research Institute, Tsukuba, Japan, mtani@kenken.go.jp
2 Senior Research Engineer, Building Research Institute, Tsukuba, Japan, t_mukai@kenken.go.jp
3 Graduate Student, Tokyo Institute of Technology, Yokohama, Japan, ogura.m.ab@m.titech.ac.jp
4 Ph.D Candidate, Tokyo Institute of Technology, Yokohama, Japan, taleb_rafik@yahoo.fr
5 Professor, Tokyo Institute of Technology, Yokohama, Japan, kono@serc.titech.ac.jp
The purpose of this study is to obtain fundamental data of relation between deformation and damage of non-structural R/C wall. First, failure mode of non-structural R/C walls shown in Fig.1 is discussed based on flexural and shear capacity estimation by design equations commonly used in Japan. Based on the calculated results, the loading test on full-scale specimens modelled as a non-structural R/C wall in actual building is implemented to reproduce the actual observed damage experimentally.

**CASE STUDY ON ACTUAL DAMAGED WALLS**

Failure mode of the non-structural walls which suffered severe damage by the 2011 Tohoku Earthquake is discussed based on the flexural and shear capacity estimation. The target non-structural walls for this case study are mullion walls with spandrel walls in the building A and B reported by NILIM and BRI (2012), and mullion wall of balcony in the building C reported by Ojio et al. (2013). However, the building C was designed by the current building code which was revised significantly in 1981, it was reported that all three walls failed in shear after the earthquake as shown in Photo.1.

The configuration and detail of three target walls are summarized in Table.1. All walls did not have confined boundary element. Only one or two longitudinal bars whose diameter was larger than those of web reinforcement were provided on the wall edges as reinforcement for openings. Flexural capacity \( Q_{mu} \) and shear capacity \( Q_{su} \) were calculated by Eqs.(1) and (2) (MLIT et al., 2007). Shear cracking capacity \( Q_{sc} \) was calculated by Eq.(3) shown in the AIJ design guideline (2001) for column with rectangular section. Contra flexure point was assumed to locate at the center of wall height. Based on “the Standard for Seismic Evaluation of Existing Reinforced Concrete Building” (JBDPA, 2005), nominal yield strength plus 49 N/mm\(^2\) was used as the yield strength of steel.

\[
Q_{mu} = \left( a_t f_y l_w + 0.5 a_{wV} f_{wV} l_w + 0.5 N D \right) / a
\]  

\[Q_{su} = \left\{ \frac{0.068 p_{we}^{0.23} (f'w_{scp} + 18)}{\sqrt{M/(QD) + 0.12}} + 0.85 f_{wH} p_{we} + 0.1 \sigma_0 \right\} t \times j
\]  

\[Q_{sc} = \phi \left( \sqrt{\sigma_r^2 + \sigma_t^2} \right) t \times D / \kappa
\]

Where, \( a_t, f_y \): area and yield strength of longitudinal reinforcement in the wall boundary area, \( a_{wV}, f_{wV} \): area and yield strength of longitudinal web reinforcement, \( l_w \): length between the centers of boundary columns (0.9D for rectangular cross-section), \( N \): axial load, \( a \): shear span length, \( p_{we} \): equivalent tensile reinforcement ratio (%), \( f'w_{scp} \): effective length of wall (=0.95D), \( f'c \): concrete compressive strength, \( M/(QD) \): moment-to-shear ratio (1 \( \leq M/(QD) \leq 3 \)), \( f_{wH} \): yield strength of horizontal web reinforcement, \( p_{we} \): horizontal web reinforcement ratio, \( \sigma_0 \): average axial stress to gross
cross-sectional area, $t_e$: equivalent wall thickness, $j$: length of rebar arm ($=7/8d$), $\phi$: capacity coefficient ($=1.0$), $\sigma_T$: concrete tensile strength ($=0.33\sqrt{f'_c}$), $D$: wall length, $\kappa$: shape coefficient ($=1.5$).

Calculated flexural, shear, and shear cracking capacities of the target walls are plotted against the axial load ratio $\eta (=N/(f'_c A_w))$, $A_w$: gross cross-sectional area of wall) in Fig.1. In the case horizontal web reinforcement is round bar or it does not have hook anchorage, horizontal web reinforcement is not supposed to be able to contribute to shear capacity because of poor anchorage and bond. Therefore, the calculated shear capacities on the assumption that horizontal web reinforcement ratio equals to be zero are also shown in Fig.1 as $Q_{su(pwh0)}$. As seen in Fig.1, all walls were predicted to fail in flexure under no axial load because their shear capacities exceeded their flexural capacities regardless of the consideration of horizontal web reinforcement contribution. In actual building, axial compression load on mullion wall is thought to be caused by vertical confinement from surrounding structural frame. Ojio et al. (2013) conducted using finite element analysis for the mullion wall of Building C considering the vertical confinement and reported that the value of axial compression load was raised with the increasing of wall lateral deformation and the similar failure mode with the actual wall was observed. As seen in Fig.1, the large/small relation of flexural capacities and shear capacities was reversed under high axial compression load. Hence, certain level of axial compression load is assumed to have been applied to the actual damaged mullion walls.

**OUTLINE OF EXPERIMENT**

Based on the discussions in the previous case study, loading test on the specimens which were designed as mullion walls of balcony was conducted to reproduce the actual damage shown in Photo.1. The main purpose of this experiment is to investigate how the experimental parameters, such as axial compression load, detail of horizontal web reinforcement and shear span ratio, affect to failure mode and damage state. Furthermore, the detail damage measurement was also conducted.

Four full-scale R/C mullion wall specimens were constructed in this experiment. Specifications and details of the specimens are shown in Table.2 and Fig.2, respectively. Experimental parameters of this experiment were axial load, detail of horizontal web reinforcement, and shear span length. Two
specimens, NSW1 and NSW2, were modelled as actual mullion wall and the difference between NSW1 and NSW2 was the axial load level. Axial load was not applied on NSW1 to simulate the usual design assumption that non-structural wall doesn’t support any axial load. On the other hand, axial compression load which assumed vertical confinement by surrounding frame was applied to NSW2. In the past, Mizutani et al. (1995) reported the experimental results for R/C wall specimens with single layer web reinforcement which showed that shear behaviour, such as maximum capacity and horizontal web reinforcement stress at maximum capacity, could be improved by anchorage methods (weld, hook and straight anchorage) in the case of specimens with relatively large aspect ratio ($L/D=3.0$). Based on this observation, other two specimens, NSW3 and NSW4, were provided twice the amount of horizontal web reinforcement and 180 degree hook anchorage of horizontal web reinforcement to assess the improvement of shear behaviour and damage, while the specimens in this experiment had smaller aspect ratio of 2.0. The shear span of NSW4 was twice spans of other three specimens. Axial compression load of 458 kN which was equivalent to axial load ratio of 0.15 was applied on NSW2, NSW3 and NSW4. Wall cross-section was rectangular of 120 mm by 1050 mm, and wall clear span was 2100 mm. D10 (SD295A) mild steel bars were used for web reinforcement with single layer. The spacing of longitudinal web reinforcement was 250 mm in all specimens. The spacing of horizontal web reinforcement was 250 mm (without hook anchorage) for NSW1 and NSW2, and 125 mm (with 180 degree hook anchorage) for NSW3 and NSW4, respectively. Two D13 (SD345) mild steel bars were provided on the wall edges as reinforcement for openings. Ready-mixed concrete with normal portland cement was used for all specimens. Material test results of concrete and reinforcement are shown in Tables 3 and 4, respectively.

Figure 3 illustrates the loading system. The assigned axial load was applied by two vertical 4000 kN hydraulic jacks and kept constant. The upper and lower stubs were kept parallel for NSW1, NSW2 and NSW3. The contra-flexure point was kept on the location of the wall top for NSW4. Lateral load was applied by horizontal 3000 kN hydraulic jack and controlled by drift angle $R$ derived by dividing relative horizontal displacement between the upper and lower stubs by wall clear height 2100 mm. Deflection controlled reversed cyclic loading consisting of one full cycle to drift angle $R$ of $\pm 1/1000$ ($\pm 0.125\%$) rad. and two full cycles to each drift angle $R$ of $\pm 1/400$ ($\pm 0.25\%$) rad., $\pm 1/200$ ($\pm 0.5\%$) rad., $\pm 1/333$ ($\pm 0.75\%$) rad., $\pm 1/100$ ($\pm 1.0\%$) rad., $\pm 1/67$ ($\pm 1.5\%$) rad. and $\pm 1/50$ ($\pm 2.0\%$) rad. were imposed. Reversed cyclic loading consists of five full cycles was imposed when capacity deterioration was observed, to check the degree of capacity deterioration by many times cyclic loading.

Steel strain was measured by strain gauges attached on the reinforcement for openings and web reinforcement as shown in Fig. 1. For separating flexural, shear and axial deformation, test area was divided vertically into four zones (between 0 mm and 550 mm for Z1, 550 mm and 1050 mm for Z2, 1050 mm and 1550 mm for Z3, and 1550 mm and 2100 mm for Z4), and displacement transducers were installed on each zone. Detail damage quantity measurement also conducted in this experiment.

### Table 2. Specifications of specimens

<table>
<thead>
<tr>
<th></th>
<th>NSW1</th>
<th>NSW2</th>
<th>NSW3</th>
<th>NSW4</th>
</tr>
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<tbody>
<tr>
<td>$t$ (mm) x $D$ (mm)</td>
<td>120 x 1050</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Clear span length (mm)</td>
<td></td>
<td>2100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear span length (mm)</td>
<td></td>
<td>1050</td>
<td>2100</td>
<td></td>
</tr>
<tr>
<td>Reinforcement for openings</td>
<td>2-D13 (SD345, $a_c=253.4$ mm$^2$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal web reinforcement</td>
<td>D10@250 (SD295A, $p_{sw}=0.24%$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Horizontal web reinforcement</td>
<td>D10@250 (SD295A, $p_{wh}=0.24%$)</td>
<td>D10@125 (SD295A, $p_{wh}=0.48%$)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design concrete strength (N/mm$^2$)</td>
<td>24</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Axial load ratio $\eta$</td>
<td>0</td>
<td></td>
<td>0.15 (=458 kN)</td>
<td></td>
</tr>
</tbody>
</table>

### Table 3. Material test results of concrete

<table>
<thead>
<tr>
<th>$f'_c$ (N/mm$^2$)</th>
<th>$\varepsilon_c$ (%)</th>
<th>$E_c$ (kN/mm$^2$)</th>
<th>$f_t$ (N/mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24.2</td>
<td>0.182</td>
<td>26.3</td>
<td>2.46</td>
</tr>
</tbody>
</table>

where, $f'_c$: concrete compressive strength, $\varepsilon_c$: strain at compressive strength, $E_c$: 1/3 $f'_c$ secant modulus of elasticity, $f_t$: tensile strength

### Table 4. Material test results of reinforcement

<table>
<thead>
<tr>
<th></th>
<th>$f_y$ (N/mm$^2$)</th>
<th>$f_u$ (N/mm$^2$)</th>
<th>$E_s$ (kN/mm$^2$)</th>
<th>$\varepsilon_y$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D10 (SD295A)</td>
<td>347</td>
<td>484</td>
<td>190</td>
<td>0.183</td>
</tr>
<tr>
<td>D13 (SD345)</td>
<td>360</td>
<td>527</td>
<td>190</td>
<td>0.189</td>
</tr>
</tbody>
</table>

where, $f_y$: yield strength of steel, $\varepsilon_y$: yield strain of steel, $f_u$: tensile strength of steel, $E_s$: elastic modulus of steel
The outline of measurement is described later.

**GENERAL OBSERVATION**

The shear force - drift angle hysteresis curve of each specimen is shown in Fig.4. Photos at the end of the loading test are depicted in Photo.2. The filled circles, triangles are diamonds in Fig.4 indicate the yielding point of reinforcement for openings, vertical wall reinforcement and horizontal wall reinforcement, respectively. The filled squares in Fig.4 indicate the points of maximum lateral capacity. Reinforcement for openings and longitudinal web reinforcement of all specimens yielded during the cycle of \( R = \pm 1/400 \) rad. or \( \pm 1/200 \) rad. before horizontal web reinforcement yielding. Therefore, all specimens were seems to have failed in flexural yielding, however each specimen showed much different deformation capacity after flexural yielding. General observation of each specimen is described below.

**NSW1:** The maximum lateral loading capacities appeared during the cycle of \( R = \pm 1/200 \) rad. Critical deterioration in lateral loading capacity was not observed until the cycle of \( R = \pm 1/67 \) rad. During the second cycle of \( R = \pm 1/67 \) rad., concrete crushing at the wall base developed, and opening and sliding of flexural cracks and flexure-shear cracks around the wall top became significant. Concrete crushing at crack surfaces near the extreme compression fiber was also observed as shown in Photo.2(a). At the cycle of \( R = \pm 1/50 \) rad., cyclic loading consisting of five full cycles was imposed after lateral loading capacity decreased significantly compared to the one at the previous \( R = \pm 1/67 \) rad. cycle. Lateral loading capacity decreased by approximately 38% in mean of positive and negative during five times cyclic loading. Failure mode was assumed to be shear failure after flexural yielding.

**NSW2:** The maximum lateral loading capacities were observed during the cycle of \( R = \pm 1/200 \) rad. and \( R = \pm 1/133 \) rad. During the cycle of \( R = \pm 1/200 \) rad., shear cracks connected two compression zones diagonally. As deformation became larger, these diagonal cracks repeated opening and closing with sliding and concrete crushing occurred at their interfaces. At the cycle of \( R = \pm 1/100 \) rad., the lateral loading capacity deterioration was observed, and cyclic loading consisting of five full cycles was imposed. Axial load carrying capacity was lost during the fifth negative cycle due to crushing around the intersection of shear cracks at the upper part. The loss of lateral loading capacity was approximately 39% in mean of positive and negative. However the failure mode was assumed to be
shear failure after flexural yielding, the same as NSW1, NSW2 showed shear-dominant behaviour and similar failure mode as seen in the actual damaged mullion walls.

**NSW3**: The maximum capacities appeared during the cycle of \( R = \pm 1/100 \) rad. Diagonal shear cracks as seen in NSW2 occurred during cycle of \( R = \pm 1/200 \) rad. After that, they didn’t developed as NSW2, and flexure-shear cracks and concrete crushing at the wall top developed. During the first cycle of \( R = 1/67 \) rad., significant loss of lateral capacity due to sliding at the flexure-shear crack around the wall top as shown in Photo.2(c). The improved behaviour is attributed to the closer spacing and hook anchorage of horizontal web reinforcement.

**NSW4**: The maximum capacities appeared during the cycle of \( R = +1/67 \) rad. and \( R = -1/100 \) rad. After that, concrete crushing at the wall base developed. During the first cycle of \( R = -1/50 \) rad, capacity deterioration was observed. Therefore, reversed cyclic loading consisting of five full cycles was conducted at this cycle. During the fifth negative cycle, concrete crushing developed near the

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**Figure 4. Relationships between shear force and drift angle**

(a) NSW1  (b) NSW2  (c) NSW3  (d) NSW4

**Photo 2. Appearance at the end of the loading test**
centroid of cross-section at the wall base, and then significant capacity loss occurred during five times cyclic loading by approximately 43%. Final failure mode was flexural compression failure. The deformation capacity was improved from NSW3 because of smaller shear force input by cantilever loading.

**DISCUSSION ABOUT CAPACITY AND FAILURE MODE**

Capacity and failure mode of each specimen are discussed by comparing capacities obtained experimentally and theoretically, ratio of flexural and shear component to total deformation and axial strain. Experimental maximum lateral loading capacity \( Q_{exp} \), calculated flexural capacity \( Q_{mu} \) by Eq.(1), calculated shear capacity \( Q_{su} \) by Eq.(2), calculated shear cracking capacity \( Q_{sc} \) by Eq.(3) are shown in Table.5. And, Fig.5 shows the ratios of flexural and shear component to total wall deformation at the peak of the first positive cycle.

The maximum lateral capacity of NSW1 was 1.09 times of calculated flexural capacity and much smaller than calculated shear capacity. As seen in Fig.5, ratio of flexural component to total displacement was around 70% by the cycle of \( R = \pm 1/200 \) rad. in which maximum lateral capacity appeared. These relatively large ratios of flexural component are consistent with flexure-dominant behaviour observed before reaching maximum capacity in the experiment. Increasing of shear component ratio as loading progress corresponds to the observation that horizontal web reinforcement yielded and then opening and closing of flexural and flexure-shear cracks with sliding at crack interfaces became significant in post-peak region. The maximum lateral capacity of NSW2 exceeded \( Q_{su(pwh0)} \) by approximately 22%, while it was smaller than \( Q_{su} \) by approximately 9%. When the maximum capacity appeared in positive loading, tensile strain of upper bar of three horizontal bars with strain gauges was 0.17%, and tensile strain of other two was smaller than 0.05%. Even at the negative maximum capacity, only upper bar yielded and tensile strain of other two was 0.15%. It notes that horizontal web reinforcement could considerably contribute to shear capacity in spite of poor anchorage. From Fig.5, shear component makes up approximately 45% to 60% of total deformation. It shows that shear behaviour was the most dominant of all specimens as observed in the experiment. The maximum lateral capacity of NSW3 exceeded that of NSW2 by approximately 9%. This improvement is assumed to be resulted mainly from the contribution of twice the amount of horizontal web reinforcement. The upper and middle horizontal bars yielded and tensile strain of lower bar was 0.11% at the positive maximum lateral capacity, and three horizontal bars yielded at the negative maximum lateral capacity. Therefore, horizontal web reinforcement is assumed to have worked more effectively compared with the case of NSW2 due to hook anchorage. However, maximum lateral capacity of NSW3 was smaller than the calculated flexural capacity by approximately 5%. As seen in Fig.5, shear component ratio of NSW3 ranged between approximately 30% and 40%. This smaller shear component is attributed to twice the amount of horizontal web reinforcement and hook anchorage which controlled the shear cracking development. The maximum lateral capacity of NSW4 was 1.05 times of calculated flexural capacity and smaller than calculated shear capacity. Flexural component accounted for more than 80% of total deformation as shown in Fig.5. These large ratios of flexural component are consistent with flexure-dominant behaviour as with NSW1.

<table>
<thead>
<tr>
<th></th>
<th>( Q_{exp} ) (kN)</th>
<th>( Q_{mu} ) (kN)</th>
<th>( Q_{su} ) (kN)</th>
<th>( Q_{su(pwh0)} ) (kN)</th>
<th>( Q_{sc} ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSW1</td>
<td>155.0</td>
<td>137.5</td>
<td>126.7</td>
<td>279.8</td>
<td>198.9</td>
</tr>
<tr>
<td></td>
<td>-120.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NSW2</td>
<td>296.6</td>
<td>289.2</td>
<td>332.8</td>
<td>317.9</td>
<td>237.0</td>
</tr>
<tr>
<td></td>
<td>-281.7</td>
<td></td>
<td></td>
<td></td>
<td>221.0</td>
</tr>
<tr>
<td>NSW3</td>
<td>321.2</td>
<td>316.4</td>
<td>332.8</td>
<td>351.3</td>
<td>237.0</td>
</tr>
<tr>
<td></td>
<td>-311.6</td>
<td></td>
<td></td>
<td></td>
<td>221.0</td>
</tr>
<tr>
<td>NSW4</td>
<td>187.1</td>
<td>174.4</td>
<td>166.4</td>
<td>297.0</td>
<td>182.7</td>
</tr>
<tr>
<td></td>
<td>-161.7</td>
<td></td>
<td></td>
<td></td>
<td>221.0</td>
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As shown in Table 5, the maximum capacities of all specimens equal to or exceed shear cracking capacities. This result is consistent with the observation that shear cracks occurred during the loading test. In general, wall fails by occurrence of shear cracking (diagonal tension failure) when the amount of wall horizontal reinforcement is low. In this experiment, significant capacity deterioration by shear cracking was not observed and every specimen maintained at least its capacity after shear cracking. It is supposed that diagonal tension failure did not occur.

Average axial strain of test area at peak of each cycle is shown in Fig. 6. Average axial strain was derived from the measured displacement by transducers installed on the wall shown in Fig. 2(a). In NSW1 with no axial load, axial elongation occurred with increasing lateral deformation as seen in Fig. 6. This axial elongation will be restrained by surrounding structural frame in the actual building. On the contrary, in NSW2 with constant axial compression load of $\eta=0.15$, axial compression deformation occurred. Though axial compression load is thought to vary according to lateral deformation actually, constant axial compression load was applied assuming severe condition. As a result, NSW1 and NSW2 showed quite different failure mode, and similar failure mode as the actual

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**Figure 5. Ratios of flexural and shear deformation to total deformation**

![Graph showing ratios of flexural and shear deformation to total deformation](image)

**Figure 6. Average longitudinal strain**

![Graph showing average longitudinal strain](image)
damage was observed in NSW2 with axial compression load. This experimental result implies that considerable axial compression load was applied in the actual damaged buildings. The magnitude of axial compression load by confinement of surrounding structural frame is thought to depend on the specifications of surrounding frame. It has to be assessed, for examples, by analytical discussion about whole building including R/C non-structural walls. NSW3 and NSW4 which had improved horizontal web reinforcement behaved similarly with NSW2 by the cycle of $R=1/200$ rad. After that, in contrast with NSW2, axial tension displacement increased monotonically in NSW3 and NSW4. If deformation capacity is improved, the larger axial compression load is possible to apply on wall in actual buildings, and capacities and failure modes of surrounding frame also have to be checked.

OUTLINE AND RESULTS OF DAMAGE QUANTITY MEASUREMENT

Damage quantity measurement targets in this experiment were crack width, crack length and area of concrete spalling. Measurement was conducted on three faces, façade of 1050 mm by 2079 mm and both side surfaces of 120 mm by 2079 mm. The region of 10.5 mm from the wall top and wall base were eliminated from measurement. Crack width measurement was conducted at peak displacement and after unloading at each cycle for all visible cracks by using crack scale with 0.05 mm intervals. The maximum width of every visible crack was recorded. If two distinct cracks connected to one crack, they were treated as one crack. In the case that one crack branched off or met at the middle of another crack, they were considered as two distinct cracks. For crack pattern diagrams, residual visible cracks after unloading of 2nd cycle of each target drift angle (1st cycle for $R=1/800$ rad.) were recorded by tracing on the transparent films (overhead projector films) lapped over the surface of specimens. Dimension of concrete spalling was also recorded by the same manner. Crack diagrams and dimension of concrete spalling recorded on the transparent films were converted into electronic data by using CAD software, and length of crack and area of concrete spalling were calculated.

Crack pattern diagrams after unloading of the cycle in which the maximum capacities appeared are shown in Fig.7. Blue and red lines show the cracks appeared in positive and negative loading, respectively. Black lines indicate the cracks occurred by loading system troubles. Many flexural and flexure-shear cracks occurred on NSW1, NSW3 and NSW4, though diagonal shear cracks and concrete crushing at the wall top could be seen on NSW3. NSW2 showed different behaviour from NSW3 as concrete crushing progressed at diagonal shear crack interfaces. As shown in Fig.7, the cracks by loading system troubles occurred as flexural cracks on NSW2 which showed shear-dominant behaviour. On NSW4, they occurred around the wall top where flexural moment was small. Consequently, these cracks are assumed to have not affected much to the structural behaviour.

Transitions of maximum crack width at peak deformation and after unloading are shown in Fig.8. Width of cracks which occurred by loading system trouble is distinguished by using open circles and squares as reference. Cracks of NSW1 and NSW2 did not tend to close after unloading. No axial load and shear cracks are assumed to have affected to this tendency. Residual crack width of NSW3 and NSW4, in which flexural cracks were dominant, tended to become less than half of crack width at peak. Difference in tendency of relationship between crack width at peak deformation and after unloading was observed according to axial load and kind of crack.

Figure 7. Crack pattern diagrams after unloading
Transitions of crack length per unit area after negative unloading are shown in Fig.9. The measured results on three faces and hinge region (between 10.5 mm and 307.5 mm from top and base critical sections for NSW1, NSW2 and NSW3, between 10.5 mm and 307.5 mm from base critical section for NSW4) are illustrated in Fig.8(a) and 8(b), respectively. In the figure, cracks are classified by residual crack width \( W_r \) (\( W_r < 0.2 \) mm, \( 0.2 \text{ mm} \leq W_r < 1.0 \) mm, \( 1.0 \text{ mm} \leq W_r < 2.0 \) mm, \( 2.0 \text{ mm} \leq W_r < 5.0 \) mm, \( 5.0 \text{ mm} \leq W_r \)). Cracks occurred by trouble of loading system were not considered.

As seen in Fig.9, both crack lengths of three faces and hinge region in all specimens increased monotonously as experienced maximum rotation angle became larger, though difference appeared in breakdown of each crack width component between four specimens. Only in the case of NSW1, the length of \( W_r < 0.2 \) mm decreased and the length of relatively wide cracks as \( W_r \geq 0.2 \) mm increased after the cycle of \( R = 1/400 \) rad. Limited cracks are thought to have developed excessively because spacing of horizontal web reinforcement was large and axial load was zero. For NSW2, big difference was not observed between the values of crack length of three faces and hinge region as shown in Fig.9 because many shear cracks occurred outside of hinge region. As to two specimens with twice the amount of horizontal web reinforcement and hook anchorage, crack length of NSW3 was almost the same as NSW1 and NSW2, while crack length of NSW4 was reduced to approximately one third of NSW1 and NSW2. However, crack widths of most cracks were less than 0.2 mm as seen in Fig.9. So, it can be said that detail of wall horizontal reinforcement could control damage.

Figure 9(a). Crack length of three faces after unloading

Figure 8. Maximum crack width at peak deformation and after unloading

Transitions of crack length per unit area after negative unloading are shown in Fig.9. The measured results on three faces and hinge region (between 10.5 mm and 307.5 mm from top and base critical sections for NSW1, NSW2 and NSW3, between 10.5 mm and 307.5 mm from base critical section for NSW4) are illustrated in Fig.8(a) and 8(b), respectively. In the figure, cracks are classified by residual crack width \( W_r \) (\( W_r < 0.2 \) mm, \( 0.2 \text{ mm} \leq W_r < 1.0 \) mm, \( 1.0 \text{ mm} \leq W_r < 2.0 \) mm, \( 2.0 \text{ mm} \leq W_r < 5.0 \) mm, \( 5.0 \text{ mm} \leq W_r \)). Cracks occurred by trouble of loading system were not considered.

As seen in Fig.9, both crack lengths of three faces and hinge region in all specimens increased monotonously as experienced maximum rotation angle became larger, though difference appeared in breakdown of each crack width component between four specimens. Only in the case of NSW1, the length of \( W_r < 0.2 \) mm decreased and the length of relatively wide cracks as \( W_r \geq 0.2 \) mm increased after the cycle of \( R = 1/400 \) rad. Limited cracks are thought to have developed excessively because spacing of horizontal web reinforcement was large and axial load was zero. For NSW2, big difference was not observed between the values of crack length of three faces and hinge region as shown in Fig.9 because many shear cracks occurred outside of hinge region. As to two specimens with twice the amount of horizontal web reinforcement and hook anchorage, crack length of NSW3 was almost the same as NSW1 and NSW2, while crack length of NSW4 was reduced to approximately one third of NSW1 and NSW2. However, crack widths of most cracks were less than 0.2 mm as seen in Fig.9. So, it can be said that detail of wall horizontal reinforcement could control damage.
Transitions of concrete spalling area per unit area after negative unloading are shown in Fig.10. As seen in Fig.10, there was much of a difference between each specimen. At the final loading cycle, approximately 55% of concrete in surface area spalled in hinge region of NSW4, while only about 2% spalled in hinge region of NSW3. The values of spalling area in hinge region of all specimens except NSW2 were larger than those in all three faces. Most of spalling area in hinge region was due to concrete crushing at compression zone. On the contrary, in the case of NSW2, difference between the values of spalling area in hinge region and in three faces was small because of concrete crushing at shear crack interface outside hinge region. The slight increasing in values of NSW2 and NSW3 at the cycle of $R=1/200$ rad. came from occurrence of concrete crushing at wall top while concrete crushing was not observed at the same cycle in NSW4 which was applied the same axial compression load. As seen in Photo.2, damage of three specimens under double-curvature loading tended to be larger at the wall top. Casting concrete in vertical direction is thought to have caused concrete strength irregularity in wall vertical direction, and it could affect failure mode and crack propagation.

CONCLUSIONS

Case study on three actual R/C non-structural walls which were damaged in the 2011 Tohoku Earthquake was conducted. And, loading test on four full-scale R/C mullion wall specimens was
implemented for the purpose of discussion about failure mode, capacity, effect of detail improvement, and damage quantity. Obtained findings are listed below.

- In the case study, all three walls were predicted to fail in flexure under no axial load. The large/small relation of flexural capacities and shear capacities was reversed under high axial compression load. Hence, certain level of axial compression load is assumed to have been applied to the actual damaged mullion walls.

- Two specimens, NSW1 and NSW2, had specifications simulated actual mullion wall. Specimen NSW1 under no axial load showed flexure-dominant behaviour and relatively good deformation capacity. On the contrary, specimen NSW2 under axial compression load of $\eta = 0.15$ showed similar failure mode as seen in the actual buildings and lost its axial load carrying capacity at relatively small lateral deformation ($\Delta = 1/100$ rad.).

- Another two specimens, NSW3 and NSW4, had better horizontal web reinforcement detail. Development of shear cracking was controlled and maximum lateral capacity and deformation capacity were improved by twice the amount of horizontal web reinforcement and hook anchorage. Cantilever loading for NSW4 also improved deformation capacity because of smaller shear force input.

- In three specimens which were imposed cyclic loading consisting of five full cycles after lateral loading capacity started decreasing, lateral loading capacities deteriorated by approximately 40% in mean of positive and negative during five times cyclic loading.

- In NSW2, measured strain of horizontal bars without hook anchorage exceeded or was close to yield strain at maximum lateral capacity. It notes that horizontal web reinforcement could considerably contribute to shear capacity in spite of poor anchorage.

- Damage quantity measurement for crack width, crack length and concrete spalling area was conducted to assess the effect of experimental parameters, such as axial load, horizontal web reinforcement and shear span, to damage state. It was shown that increasing the amount of horizontal web reinforcement and installing hook anchorage could control development of shear cracking and damage quantity after unloading.

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