Seismic Performance Evaluation of RC Moment-Resisting Frames with Typical Non-Structural Walls in Japan

Rokhyun Yoon, Yasushi Sanada, Takumi Akahori

Journal of Advanced Concrete Technology, volume 15 (2017), pp. 544-557

Related Papers [Click to Download full PDF!](#)

A New Methodology for Performance-based Seismic Evaluation of Low-rise Reinforced Concrete Buildings Using Nonlinear Required Strengths

Kang Seok Lee, Sung Woo Shin

Journal of Advanced Concrete Technology, volume 11 (2013), pp. 151-166

Shear Performance of RC Columns with Wide Hoop Spacing at Mechanical Splices

Syafri Wardi, Yasushi Sanada, Yuebing Li, Hiroshi Kuramoto

Journal of Advanced Concrete Technology, volume 14 (2016), pp. 521-534

Experimental and Analytical Investigations of Seismic Pounding of Adjacent 14-Story Reinforced Concrete Buildings Damaged in 1985 Mexico Earthquake

Takahiro Chujo, Hiroshi Yoshikado, Yuichi Sato, Kazuhiro Naganuma

Journal of Advanced Concrete Technology, volume 14 (2016), pp. 753-769

Seismic Performance Evaluation of Masonry Infilled Reinforced Concrete Buildings Utilizing Verified Masonry Properties in Applied Element Method

Arifa Iffat Zerin, Akira Hosoda, Hamed Salem, Khan A Uva i X Amanat

Journal of Advanced Concrete Technology, volume 15 (2017), pp. 227-243

[Click to Submit your Papers](#)

Scientific paper

Seismic Performance Evaluation of RC Moment-Resisting Frames with Typical Non-Structural Walls in Japan

Rokhyun Yoon¹, Yasushi Sanada^{2*} and Takumi Akahori³

Received 23 January 2017, accepted 8 September 2017

doi:10.3151/jact.15.544

Abstract

Reinforced concrete (RC) buildings are often constructed with non-structural walls. Severe damage to non-structural walls has been observed in many RC buildings after major earthquakes in and around Japan, such as the 2011 earthquake off the Pacific coast of Tohoku. Several studies have verified that non-structural walls affect the seismic performance of RC buildings. However, no design methodology has been proposed for considering the structural effects of non-structural walls. This study focuses on the RC non-structural walls used as exterior/partition walls in typical residential buildings in Japan. Cyclic loading tests were performed using three 1/2.5 scale, one-story, one-bay RC moment-resisting frame specimens with and without non-structural walls, which were monolithically constructed or structurally isolated by seismic slits. The isolated wall as well as the monolithic wall significantly increased the strength of the moment-resisting frame specimen. Furthermore, this study proposes analytical models to simulate the experimental results and to clarify the effects of non-structural walls on the overall performance of test specimens. The test specimens were replaced by line elements with multi-spring models while considering the interaction between the bending moment and axial force. The analytical simulations generally agreed well with the experimental results. In conclusion, the analytical models applied to the simulations in this study are effective for evaluating the seismic behavior and performance of RC moment-resisting frames with the typical non-structural walls used in Japan.

1. Introduction

Reinforced concrete (RC) buildings are typically designed with consideration for their structural components. However, they are often constructed with non-structural components. Severe damage to non-structural walls has been observed in many RC buildings after major earthquakes in and around Japan, such as the 2011 earthquake off the Pacific coast of Tohoku, as shown in **Fig. 1** (AIJ 2012). Moreover, several studies have previously reported that such non-structural walls affected the seismic performance of RC buildings (e.g., Ju *et al.* 2012; Kabeyasawa *et al.* 2014; Orakcal *et al.* 2009). However, no common design methodology has been proposed that considers the structural effects of the non-structural walls.

RC non-structural walls include a wide range of components. Wing, hanging/spandrel, and mullion walls monolithic to structural members had been typical in/before the 1990s in Japan, while isolated walls from primary moment-resisting frames have become more popular particularly after the 1995 Kobe earthquake. According to the latest design standards for RC buildings in Japan (AIJ 2010a), RC walls satisfying several

requirements such as the minimum thickness (≥ 120 mm) and rebar details (reinforcement ratio $\geq 0.25\%$ etc.) are regarded as structural components, when they are designed to resist external loads. Although other walls

Fig. 1 Photographs of damage to non-structural walls.

¹JSPS Research Fellow, Graduate School of Engineering Osaka University, Suita, Japan.

²Associate Professor, Graduate School of Engineering, Osaka University, Suita, Japan. *Corresponding author, E-mail: sanada@arch.eng.osaka-u.ac.jp

³Former Graduate Student, Graduate School of Engineering, Osaka University, Suita, Japan.

unsatisfying such requirements are generally neglected in structural design, they may be considered to evaluate captive-column effects under engineering judgement. In contrast, seismic gaps are often applied between RC walls and primary frames to avoid unexpected wall-frame interactions in recent design, as introduced later. In particular, once RC walls are regarded as non-structural walls, their effects are apt to be overlooked; thus, unexpected wall effects must be studied to clarify more exact seismic performance of RC buildings.

Recently, a shake table test was conducted using the world's largest three-dimensional shake table (E-Defense) in Japan to clarify the potential margins of collapse for typical RC residential buildings designed according to the current building design standards in Japan (Sugimoto *et al.* 2017). The experimental data of the shake table test have been opened to the public from E-defense. For a preliminary investigation, the authors performed a series of laboratory tests to obtain fundamental data on the seismic behavior of RC non-structural walls, which were used as exterior/partition walls in a prototype building for the shake table test.

Under current seismic design practice in Japan, RC non-structural walls have often been separated from RC moment-resisting frames by installing seismic slits between the non-structural wall and the primary frame, as mentioned above. The seismic slits are usually provided along three boundaries of the wall, except the wall-top beam boundary, as shown in **Fig. 2**. Hence, the wall is monolithic with the top beam. Yanagisawa (2008) tested 1/6 scale specimens and showed that separating a RC infill wall from a RC moment-resisting frame with seismic slits (the slit width was 1.8% of the wall height) could effectively maintain the behavior of the infilled moment-resisting frame, similar to that of the bare moment-resisting frame, up to a 1.8% drift ratio. Ju *et al.* (2012) tested 4/5 scale specimens and confirmed that separating the RC infill wall from the steel moment-resisting frame with slits (slit width was 1.5% of the

wall height) could effectively prevent interactions between the infill wall and the surrounding frame up to a 1.5% drift ratio. Another experimental study by Lee *et al.* (2008) on 4/5 scale RC frame specimens with a slit width of 1.5% of the wall height obtained similar outcomes. However, experimental knowledge of the effects of seismic slits is still limited because tie-bars to prevent out-of-plane deformation are applied to seismic slits according to the Japanese construction guidelines for RC buildings (AIJ 2010b), as illustrated in **Fig. 2**, which is also discussed in the current paper.

This study is a preliminary investigation for the shake table test at E-Defense. The objective is to experimentally and analytically investigate the seismic behavior and performance of RC moment-resisting frames with and without two types of non-structural walls under static cyclic loading. Several numerical models are newly proposed and verified through comparisons between the test and simulation results. In particular, the analytical research outcomes from this study will contribute to a more precise seismic design and evaluation by considering the non-structural wall effects using recent performance-based methodologies (e.g., ATC 40 1996; ASCE/SEI 41-06 2007).

2. Experimental procedure

2.1. Details of test specimens

Figure 3 shows the typical floor framing plan and elevations of a prototype building, which is an eight-story, three-bay x one-bay, RC moment-resisting frame building that is normally used for residences in Japan. The prototype building was designed according to the current Japanese building standards (BCJ 2013). Three 1/2.5 scale, one-story, one-bay plane frame specimens with and without non-structural walls were designed to represent the 2nd-story partial frame in the span direction of the prototype building, as summarized in **Table 1**. **Figure 4** shows the dimensions and reinforcement details of the specimens. The dimensions of the columns and beams were 300 × 320 mm and 180 × 280 mm, respectively. The story height and span length were 1,200 mm and 2,250 mm, respectively. **Table 2** summarizes the combination of test parameters. One of the specimens, BF, represented the bare frame without non-structural walls. Two others had different types of non-structural walls, which were monolithically constructed and structurally isolated by seismic slits (named WF and WFs, respectively). In the case of WFs, a small number of tie-bars (D4@160) were provided along the seismic slits to prevent out-of-plane deformation of the wall according to the Japanese guidelines (AIJ 2010b), as shown in **Fig. 4c**. The cylindrical compressive strength of the concrete and the properties of the reinforcement are shown in **Tables 3** and **4**. The concrete values on **Table 3** were obtained from standard cylinder tests according to the Japan Industrial Standards (JIS A 1108 2006, JIS A 1149 2010).

Fig. 2 Typical seismic slit design.

Table 1 Specifications of members.

		Prototype building	Test specimen
Beam	$B \times D$	450 × 700	180 × 280
	Longitudinal rebar	6-D25 ($p_{tb} = 0.48$)	4-D13 ($p_{tb} = 0.50$)
	Shear reinforcement	2-D13@200 ($p_{wb} = 0.28$)	2-D5@80 ($p_{wb} = 0.31$)
Column	$B \times D$	750 × 800	300 × 320
	Longitudinal rebar	14-D25 ($p_{tc} = 0.34$)	16-D10 ($p_{tc} = 0.34$)
	Shear reinforcement	D13@100 ($p_{wc} = 0.28$)	D10@40 ($p_{wc} = 0.31$)
Wall	Thickness	180	72
	Vertical and horizontal reinforcement	D10@200double	D4@100double
	Reinforcement along the perimeter of the opening	2-D16	2-D5
	Opening ratio*	Unfixed	0.4

p_{tb} : Tensile reinforcement ratio of beam; p_{tc} : Tensile reinforcement ratio of column

p_{wb} : Shear reinforcement ratio of beam; p_{wc} : Shear reinforcement ratio of column

*Opening ratio is defined by $\eta = (h_i/l_i/h)^{1/2}$

(η : opening ratio; h_i/l_i : height/length of opening; and h/l : story height/span length) (JBDPA 2005)

(a) Typical floor framing plan

(b) Framing elevations along the axes of A-B/1-4

Fig. 3 Typical floor framing plan and elevations of a prototype building.

Table 2 Experimental parameters.

Specimen	Non-structural wall	Slit
BF	Without	Without
WF	With	Without
WFs	With	With

Table 3 Material properties of concrete.

Specimen	Compressive strength (N/mm ²)	Elastic modulus* (kN/mm ²)
BF	33.5	25.7
WF	35.5	26.5
WFs	33.4	26.5

* Elastic modulus is defined by $E = (f_c/3 - f_2)/(\epsilon_1 - \epsilon_2)$
 (E : elastic modulus; f_c : compressive strength of concrete;
 f_2 : stress at 50×10^{-6} strain; ϵ_1 : strain at $f_c/3$; ϵ_2 : 50×10^{-6}) (JIS
 A 1149 2010)

Table 4 Material properties of reinforcement.

	Yield stress (N/mm ²)	Tensile strength (N/mm ²)	Elastic modulus (kN/mm ²)
D4	320	499	168
D5	313	501	163
D10	381	519	179
D13	391	554	185

2.2. Experimental methods

The specimens were tested using the loading system at Osaka University. A test setup in the loading system is shown in **Figs. 5** and **6**. The specimens were mounted onto pin supports in the loading system, which consisted of one horizontal and two vertical hydraulic jacks, as shown in **Fig. 5**. Every specimen was subjected to a constant axial load of 576 kN ($N/N_0 = 0.1$, where N is the axial load, and N_0 is the design compressive strength

Fig. 4 Dimensions and reinforcement details of the specimens.

Fig. 5 Test set-up and loading system.

of the columns). Then, static cyclic lateral loads were applied according to the loading history shown in **Fig. 7**, keeping the loading beam in **Figs. 5** and **6** horizontal. The lateral loading was controlled by an averaged inter-story drift ratio, R (rad), as shown in **Fig. 6**.

3. Experimental results

Figures 8 and **9** compare the lateral force vs. drift ratio relationships and crack patterns among the three specimens. The symbols on the lateral force vs. drift ratio relationships represent drifts at the formation of a yield mechanism with flexural hinging at all beam ends ($\blacktriangle$), maximum strength ($\bullet$), and ultimate state with buckling of the beam longitudinal reinforcement ($\times$). **Figure 9** shows the damage to each specimen at forming the yield mechanism and after testing to be referred in the analytical modeling described later. The behavior of each specimen throughout loading is summarized in the following section.

3.1 Experimental behavior

3.1.1 Specimen BF (Bare frame)

Flexural cracks appeared at the upper and lower beam ends during the first cycle to $R = 1/800$ rad. The speci-

men formed a mechanism after flexural yielding at all beam ends in the cycle to $R = 1/200$ rad; thus, the stiffness significantly degraded in the following cycle. A lateral resistance of 71 kN at formation of a yield mechanism was observed at the peak drift in the cycle to $R = 1/133$ rad. However, the lateral resistance still increased with an increase of lateral drift up to the final loading cycle. This effect was caused by axial elongation of the beam, which was measured as shown in **Fig. 5**, with incremental lateral drift, as shown in **Fig. 10**, because reactive compression was applied to the beam cross-section due to the constraint of axial elongation by both columns. The maximum strength reached 107 kN at the peak drift of the cycle to $R = 1/25$ rad, where buckling of longitudinal rebar and spalling of the concrete cover were also observed at the beam ends.

A lateral resistance of 43 kN was calculated according to the Japanese design standards, where beam flexural strengths were obtained with **Eq. (1)** (JBDPA 2005):

$$M_u = 0.9 \cdot a_t \cdot \sigma_y \cdot d \quad (1)$$

where a_t is the cross-sectional area of the tensile reinforcing bars, d is the effective depth of the beam, and σ_y is the yield stress of the tensile reinforcing bars.

Figure 8a compares the experimental lateral force vs. drift ratio relationship with the resistance calculated according to **Eq. (1)**. The calculated result underestimated the experimental resistance. This difference was

Fig. 6 Photograph of the test set-up.

Fig. 7 Lateral loading history for the tests.

$\blacktriangle$ Lateral resistance at formation of a yield mechanism ($_{test}Q_{mcn}$); $\bullet$ Maximum strength (Q_{max}); and $\times$ Ultimate state with buckling of beam longitudinal reinforcement

Fig. 8 Lateral force vs. drift ratio relationships.

caused by the same mechanism as noted above: the axial elongation of the beam with the plastic deformation was constrained by the columns, after which axial compression was applied to the beam. As a result, the design calculation underestimated the experimental resistance because of neglecting the axial compression.

3.1.2 Specimen WF (with a monolithic non-structural wall)

The initial stiffness of WF significantly increased compared to that of BF. Flexural cracks on the concrete surface and the yielding of horizontal rebar were observed at the ends of spandrel/hanging walls during the first cycle to $R = 1/800$ rad. Shear cracks and yielding of horizontal rebar were also observed at the flat wall between openings in the same loading cycle. In the following cycle, to $R = 1/400$ rad, the spandrel/hanging walls cracked in shear, after which the flat wall yielded in

flexure with spalling of the concrete cover. The beam longitudinal rebar began to yield in the cycle to $R = 1/200$ rad. In WF, large crack openings were observed along the inclined surfaces connecting the beam end and the corner of the opening, as shown in **Fig. 9b**. The specimen seemed to form a mechanism in the following cycle to $R = 1/133$ rad because of a significant degradation of the stiffness. Lateral resistance at the mechanism formation was 268 kN, reaching the maximum strength. This resistance was approximately 3.8 times that of BF.

3.1.3 Specimen WFs (with a non-structural wall isolated by seismic slits)

Specimen WFs behaved in a similar manner to that of BF until the cycle to $R = 1/133$ rad, while the initial stiffness of WFs increased compared to that of BF. Flexural cracks occurred at the beam ends during the first cycle to $R = 1/800$ rad. However, damage to the

Fig. 9 Front views of crack patterns.

Fig. 10 Axial elongation vs. drift ratio relationships for the upper beams.

upper beam was locally observed at the beam ends with a length approximately equivalent to the beam depth, which seemed to be affected by the presence of the non-structural wall. A lateral resistance of 103 kN, which was approximately 1.5 times that of BF, was recorded at the formation of a yield mechanism during the cycle to $R = 1/133$ rad. Then, the lateral resistance of the specimen rapidly increased after the bottom corner of the non-structural wall contacted the column in the cycle to $R = 1/67$ rad, as shown in **Figs. 8c** and **11**. As a result, shear cracks occurred on the flat wall between the openings. The maximum strength reached 172 kN at the peak drift of the cycle to $R = 1/33$ rad, where buckling of longitudinal rebar was also observed at the beam ends. The compressive failure of concrete was significant at the beam ends and the corners of openings in the cycle to $R = 1/25$ rad, causing a reduction in the strength of the specimen. Focusing on the tie-bars that were arranged at the wall-frame boundaries, they yielded in tension at vertical and horizontal slits during the first cycle to $R = 1/800$ rad.

3.2 Curvature of the upper beam

The flexural cracks at each beam end in BF and WF occurred approximately within a length of twice the beam depth from the beam end, as shown in **Fig. 9**. However, the flexural cracks on the upper beam in WFs were observed at both ends within one length of the beam depth, whereas those on the lower beam in WFs occurred in a similar manner to BF. As noted above, the non-structural wall that was isolated by seismic slits may influence the plastic length at the beam ends.

Figure 12 shows the curvature profiles along the upper beam from $R = 1/400$ rad to $R = 1/133$ rad. The curvatures were calculated by using the outputs of strain gauges on the longitudinal rebar in the beam, as shown in the figure. As indicated in the figure, significant increases of the curvature in WF were more widely distributed than those in BF and WFs, which was likely affected by the presence of the monolithic hanging wall. However, the area with high curvature in WFs was similar to that of BF despite the different crack patterns at the beam ends between both specimens.

(a) Overall view

(b) Close-up of the bottom corner of the wall

Fig. 11 Photograph of WFs at the cycle to $R = 1/67$ rad.

Fig. 12 Curvature profiles along the upper beam.

Moreover, plastic zone was investigated in **Fig. 12**, resulting in the shaded zones where the observed curvatures approximately reached/exceeded the yield curvatures with yielding of tensile rebar. As a result, the plastic zones in the cases of BF and WFs were approximately half of the beam depth, whereas that of WF was approximately half the depth of the beam and hanging wall. Furthermore, the figure indicates that the longitudinal rebar in the beam-column connections has not reached the yield strain.

4. Analytical modeling

4.1. Common modeling of specimens

The experimental results indicated that the structural behavior and seismic performance of the specimens must be evaluated by considering the interaction between the bending moment and axial force applied to the beams. In addition, appropriate modeling for yield surfaces in the WF and the tie-bars and collision of the non-structural wall in WFs seemed to be key issues for numerical simulations of the tests. Thus, the specimens were replaced by idealized numerical models, as shown in **Fig. 13**, to rationally account for the above experimental behavior. Moreover, in this study, pushover analyses were performed for numerical simulations of the tests because the pushover analysis is becoming a method for evaluating the seismic demands and seismic performance of new and existing structures in performance-based seismic design/evaluation methodologies such as ATC 40 (1996) and ASCE/SEI 41-06 (2007). A commercial nonlinear analysis software (SNAP 2015) was used for the analyses.

The story height and span length in the analytical models were the same as those of the specimens. However, the analytical models represented up to the inflection points of the columns in the upper and lower stories,

which were located at the center of the pin supports, as shown in **Fig. 5**. In addition, the pin supports at the ends of columns were assumed to be rigid, as shown in **Fig. 13**. For the boundary conditions, two pin supports in the upper story were configured to have the same displacements in both the x- and y-directions, while the other pin supports at the bottom were assumed to be fixed. Applied lateral loads were controlled by the inter-story drift ratio R up to 1/25 rad under the vertical loads equivalent to 10% of the gross compression capacity of the columns.

A model called the multi-spring model (MS model) for reinforced concrete columns developed by Lai *et al.* (1984) and Li *et al.* (1988) was proposed to simulate the inelastic behavior of the columns. The MS model consists of several uniaxial steel and concrete springs to represent the inelastic flexural rotation and the N-M interaction at the column ends. In this study, the MS model was applied to the beams as well as to the columns to evaluate the inelastic flexural behavior under compression. **Figure 14** shows the modeling of the member cross-sections for the MS model, where the symbols in the parentheses correspond to those in **Fig. 13**. The cross-sections were replaced by concrete elements with a depth of 20 mm and steel elements crossing the sections or an extra plane beyond the sections, as shown in **Fig. 14**.

4.2. Specific modeling of specimens

4.2.1 Specimen BF (Bare frame)

The columns and beams in the middle story were replaced by a line element consisting of two MS models at both member ends sandwiching a linear element. The critical sections of the columns and beams were defined to be at the beam and column surfaces. Rigid zones at the column and beam ends were assumed to be within a range from each node to 1/4 of the member depth. The

Fig. 13 Overall analysis models for the specimens.

plastic hinge length in the MS model was assumed to be 1/2 of the member depth based on the experimental results described in Section 3.2.

4.2.2 Specimen WF (with a monolithic non-structural wall)

The columns and beams in the mid-story were modeled by considering the wing and spandrel/hanging walls, respectively. The critical sections of the columns were defined to be at the spandrel/hanging wall ends, while those of the beams were assumed to be an oblique cross-section taking into account the experimental results, as shown in Figs. 9b and 15. As shown in Fig. 15b, the cross-sectional areas of the longitudinal and transverse bars a_l and a_t were converted into $a_l/\cos\theta$ and $a_t/\sin\theta$, respectively. The rigid zones at the member ends were modeled in the same manner as BF, considering low curvature (strain) at the center of the beam-column joints, as shown in Fig. 12. The plastic hinge length of the MS model for the beams was assumed to be 1/2 of the whole depth of the beam with the spandrel/hanging wall based on the test results, while that for the columns

was modeled in the same manner as BF, considering the short length between the critical sections. In addition, the flat wall between the openings was represented by a line element with the MS models at both ends and non-linear shear spring. The MS models at the critical sections were defined to be at the spandrel/hanging wall ends. A tri-linear model was applied to the shear spring considering strength deterioration, as shown in Fig. 16. The resistance to shear cracking Q_{cr} and the corresponding drift R_{cr} were evaluated with Eqs. (2) and (3) according to the Japanese common practice (AIJ 2010a). Q_{su} was the ultimate shear strength given by Eq. (4) (JBDPA 2005). The drift at the ultimate shear strength, R_u , was assumed to be 0.004 rad according to the Japanese standards (JBDPA 2005). The post-peak deterioration was modeled based on the experimental results reported by Sanada *et al.* (2017).

$$Q_{cr} = \left(\frac{0.085 \cdot k_c \cdot (F_c + 500)}{M/(Q \cdot D) + 1.7} \right) \cdot b \cdot j \quad (2)$$

where k_c is 0.72, F_c is the compressive strength of con-

Fig. 14 Modeling of member cross-sections for the MS model.

Fig. 15 Modeling of the critical section of specimen WF.

Fig. 16 Shear spring model considering strength deterioration.

crete, $M/(Q \cdot D)$ is the shear span-to-depth ratio, b is member depth (wall thickness), j is $7/8 \cdot d$, and d is the effective depth ($= 0.95 D$).

$$R_{cr} = Q_{cr} / K_s \quad (3)$$

where K_s is the elastic shear stiffness.

$$Q_{su} = \left(\frac{0.053 \cdot p_t^{0.23} \cdot (18 + F_c)}{M/(Q \cdot d) + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy} + 0.1 \cdot \sigma_0} \right) \cdot t_e \cdot j \quad (4)$$

where p_t is the tensile reinforcement ratio, p_w is the shear reinforcement ratio, t_e is the equivalent wall thickness, σ_{wy} is the yield stress of the shear reinforcement, and σ_0 is the axial stress.

4.2.3 Specimen WFs (with a non-structural wall isolated by seismic slits)

For specimen WFs with a non-structural wall isolated by seismic slits, the columns and the lower beam were modeled in the same manner as those for specimen BF. The upper beam was modeled with consideration for the presence of tie-bars at the vertical slits and the hanging wall above the openings. For the MS models in the upper beam ends above the vertical slits, the cross-sections were represented, including the tie-bars on the vertical slit, as shown in Fig. 14vii. In addition, the cross-section at the mid-span considered the hanging wall above the openings, as shown in Fig. 14iv. The plastic hinge length of the MS model was assumed to be equal to 1/2 of the column and beam depths, similar to those of BF, based on the test results shown in Section 3.2. However, relative horizontal displacement between the lower beam and the non-structural wall was observed, as shown in Fig. 11; thus, the tie-bars at the horizontal slit carried shear force. The tie-bars of the horizontal slit were replaced by a line element that connected the centers of the upper and lower beams. The flexible height of the line element was assumed to be equal to the clearance of the horizontal slit, considering a rigid zone along the wall panel. Bending springs with the common bilinear model, as shown in Fig. 17, were provided at the ends of the line element, as shown in Fig. 13c. The initial stiffness of the bending spring was obtained by

multiplying the number of tie-bars by the elastic bending stiffness of a tie-bar. The full plastic moment, M_u , was obtained with Eq. (5):

$$M_u = \frac{\pi \cdot d^2}{8} \cdot \sigma_y \cdot \frac{4d}{3\pi} \cdot n_s = \frac{d^3}{6} \cdot \sigma_y \cdot n_s \quad (5)$$

where d is the diameter of the tie-bar, σ_y is the yield stress of the tie-bar, and n_s is the number of tie-bars in the horizontal slit.

Furthermore, the collision between the non-structural wall and column observed in the experiment was represented by a shear spring applied to the diagonal line element in Fig. 13c. An asymmetric model was applied to the shear spring, as shown in Fig. 18. A drift angle at the collision, R_{12} , was defined with Eq. (6). The ultimate shear strength of the non-structural wall without boundary columns, nQ_{su} , was evaluated according to Eq. (4). However, this variable was reduced from Eq. (4) by multiplying the strength reduction factor γ by Eq. (7) for walls with openings in Japanese design practice (JBDPA 2005). The shear deformation drift at the ultimate shear strength, nR_u , was represented by Eq. (8).

$$R_{12} = \frac{w_s}{h_n} \quad (6)$$

where w_s is the seismic slit width ($= 12$ mm), and h_n is

Fig. 17 Restoring force model of the tie-bars in the horizontal slit.

Fig. 18 Shear spring model representing the collision of a non-structural wall.

the non-structural wall height.

$$\gamma = 1 - \eta \quad (7)$$

where η is the opening ratio (see **Table 1**).

$${}_nR_u = R_{12} + R_u = R_{12} + 0.004rad \quad (8)$$

4.3 Material Properties

Figure 19 illustrates stress-strain characteristics of concrete and reinforcements for the MS model, which are selected considering good agreements with the stress-strain relationships from material tests. The Hognestad model (Hognestad *et al.* 1955) was adopted for the envelope compressive stress-strain relationship of concrete, including the ascending and descending branches, respectively, as shown in **Fig. 19a**; thus, confining effects by transverse reinforcement were neglected for the concrete in the following analyses. The tensile behavior of the concrete was defined to be linear, ranging up to the cracking strength. In addition, a bilinear model was used for the stress-strain relationship of the reinforcements. The mechanical properties of concrete and the reinforcements were referred from the experimental results, as shown in **Tables 3** and **4**.

5. Simulation results

5.1. Lateral force-drift ratio relationship

Figure 20 compares the lateral force vs. drift ratio relationships from the experiments with the analytical results. Symbols on the lateral force vs. drift ratio relationships are as follows: drift at the initial cracking of the beam ($\blacktriangle$), flexural yielding of beam ($\bullet$), and shear failure of the flat wall ($\blacksquare$).

The analytical models for all specimens evaluated the experimental initial stiffness well. In the analytical results, however, the stiffness after flexural cracking was slightly higher than that in the experimental results, which resulted in lower drift at the flexural yielding. Such underestimates were attributed to the pullout behavior of the longitudinal rebar from the concrete in the

experiments, which was not considered in the analyses. Focusing on the analytical results of WF, the flat wall between openings reached the ultimate strength at a small drift ratio, as observed in the experiment. Moreover, the compressive failure of the concrete was observed at the corners of openings from the cycle to $R = 1/133$ rad in the MS model, representing the beams with the spandrel/hanging wall showing strength deterioration. On the other hand, focusing on the analytical results of WFs, the tie-bars, which were arranged at the wall-frame boundaries, yielded at both the vertical and horizontal slits by a drift ratio of $R = 1/800$ rad. Moreover, even after a collision between the non-structural wall and column, the experimental behavior of the specimen could be simulated well because the collision was considered by the shear spring shown in **Fig. 18**.

As noted above, the simulated performance curves of the specimens were in close agreement with the experimental envelope curves, indicating that the proposed analytical models can contribute to seismic design calculations and/or spectral response estimations (e.g., ATC 40 1996, ASCE/SEI 41-06 2007) for RC buildings with typical non-structural walls in Japan.

5.2. Beam axial elongation

Figure 21 shows a comparison of the axial elongation of the upper beam at the peak drift between the analysis and the experiment for each specimen. The analysis results were similar to the experimental results: the axial elongation of each specimen increased with an increase of lateral drift. However, the analytical values after the cycle to $R = 1/100$ rad began to underestimate the experimental values, resulting in a maximum of approximately 50% error. This effect was caused by the analytical assumptions that neglected the pullout behavior of longitudinal rebar, which was more significant in the case of WF with the larger depth of the beam with the spandrel/hanging wall. This result indicated that the applied axial compression to the beam in the analysis was smaller in comparison with that in the experiment. However, the simulated performance curves of the specimens were in close agreement with the experimen-

Fig. 19 Stress-strain relationships for concrete and steel reinforcement.

tal envelope curves, as shown in **Fig. 20**. Therefore, the axial compression applied to the beam beyond $R = 1/100$ rad seemed not to be high because of the stiffness degradation with the damage to both columns (which constrained the beam axial elongation), as shown in **Fig. 9**.

As noted above, the beam behavior in the experiment must be affected by the beam axial elongation because compression was applied to the beam with constraint to its elongation by both columns. Therefore, in conclusion, the lateral resistance of the specimens could be evaluated well using the MS model at the beam ends, as the performance curves were evaluated while considering the moment-axial force interactions.

5.3. Moment diagram from the simulation

Figure 22 shows moment diagrams for the specimens at the cycle to $R = 1/100$ rad from the analyses. The boundary columns were not subjected to antisymmetric moment diagrams in all specimens due to the axial elongation of the beam. According to the experimental results shown in **Fig. 9**, flexural cracks occurred at the exterior side of the columns, as illustrated by the estimated moment diagrams. Moreover, focusing on the columns in the upper and the lower stories, large bending moments were applied to the left column in the upper story and the right column in the lower story. In the other columns, the direction of lateral force was opposite to the loading direction. These column moment diagrams resulted from bending moments caused by the

Fig. 20 Analytical performance curves compared with the experimental results.

Fig. 21 Axial elongation vs. drift ratio relationships and comparison with the experimental results.

Fig. 22 Bending moment diagrams from the analyses at the cycle to $R = 1/100$ rad.

beam axial elongation as well as the lateral force. As indicated in **Fig. 21**, the beam axial elongation was larger in the case of WF than in BF and WFs, which might cause flexural yielding of the columns in the upper and lower stories.

For the specimen WFs, the critical section of the upper beam sustained a larger bending moment than the lower beam, which resulted from the tie-bars that were considered in the MS model. Then, the line element replacing tie-bars on the horizontal slit sustained a lateral force of 9 kN (9% of the overall lateral resistance).

6. Conclusions

This paper experimentally and analytically investigated the effects of RC non-structural walls on the seismic performance of RC moment-resisting frames with and without the seismic slits typically used in Japan. The major findings are summarized as follows:

- 1) It was experimentally verified that the RC non-structural walls used in typical residential buildings in Japan significantly increased the stiffness and strength of the RC moment-resisting frames, indicating that the non-structural wall effects should be considered in performance-based seismic design.
- 2) In particular, it was found that the seismic performance of the specimen WFs (with a seismically isolated non-structural wall) was still affected by a small number of tie-bars provided at the wall-frame

boundaries, which have been neglected in practical structural design in Japan.

- 3) By adopting the MS model to the beams in the RC moment-resisting frame specimens, the experimental behavior that was affected by interactions between the bending moment and axial force was well simulated.
- 4) The application of the MS model to oblique critical sections was proposed in this study, aiming to simulate the behavior and performance of the specimen WF, which had a monolithic non-structural wall. Consequently, the proposed model was confirmed to evaluate the experimental performance well, including post-peak deterioration.
- 5) For simulating the behavior and performance of the specimen WFs, which had an isolated non-structural wall with seismic slits, new modeling of the tie-bars arranged at the wall-frame boundaries was proposed and verified to show good agreement with the experimental results. Moreover, new modeling representing a collision between the non-structural wall and column effectively simulated the experimental post-collision behavior of the specimen.

The above research outcomes show that the analytical models proposed in this study can contribute to more precise seismic design calculations and/or spectral response estimations for RC buildings with typical non-structural walls in Japan. In particular, the structural effects of isolated walls with seismic slits can be evalu-

ated, which are not considered in current practical design in Japan because of a lack of awareness of the potential effects. However, further verification studies are recommended because the findings are based on the limited number of specimens.

Acknowledgements

This study was supported by a Grant-in-Aid for JSPS Fellows (Grant Number 15J00625), the Special Project for Reducing Vulnerability for Urban Mega Earthquake Disasters, (ii) the Maintenance and Recovery of Functionality in Urban Infrastructures, and the Japan Ministry of Education, Culture, Sports, Science and Technology (MEXT).

References

- ASCE/SEI 41-06, (2007). "Seismic rehabilitation of existing buildings." American Society of Civil Engineers.
- ATC-40, (1996). "Seismic evaluation and retrofit of concrete buildings." Applied Technology Council: Volume 1., Report No. SSC, 96-01.
- Architectural Institute of Japan (AIJ), (2012). "Preliminary reconnaissance report of the 2011 Tohoku-Chiho Taiheiyō-Ōki Earthquake." Springer: Tokyo, Japan.
- Architectural Institute of Japan (AIJ), (2010a). "AIJ Standard for structural calculation of reinforced concrete structures." Tokyo: Architectural Institute of Japan. (in Japanese)
- Architectural Institute of Japan (AIJ), (2010b). "Recommendation for detailing and placing of concrete reinforcement." Tokyo: Architectural Institute of Japan. (in Japanese)
- Japan Building Disaster Prevention Association (JBDPA), (2005). "Standard for seismic evaluation of existing reinforced concrete buildings, 2001." 1st English Edition.
- JIS A 1108, (2006). "Method of test for compressive strength of concrete." Japanese Standards Association. (in Japanese)
- JIS A 1149, (2010). "Method of test for static modulus of elasticity of concrete." Japanese Standards Association. (in Japanese)
- Ju, R. S., Lee, H. J., Chen, C. C. and Tao, C. C., (2012). "Experimental study on separating reinforced concrete infill walls from steel moment frames." *Journal of Constructional Steel Research*, 71, 119-128.
- Hognestad, E., Hanson, N.W. and, McHenry, D., (1955). "Concrete stress distribution in ultimate strength design." *ACI Journal*, 52(12), 455-480.
- Lee, H. J., Ju, R. S. and Chen, Y. L., (2008). "Experimental studies on seismic performance of building frames with infilled RC walls." Taiwan: Architectures and Building Research Institute, Ministry of the Interior 978-986-01-7136-5: Report No.097301070000G1016. (in Chinese)
- Lai, S., Will, G. T. and Otani, S., (1984). "Model for inelastic biaxial bending of concrete members." *Journal of Structural Engineering*, 110(11), 2563-2584.
- Li, K. N., Aoyama, H. and Otani, S., (1988). "Reinforced concrete columns under varying axial load and bi-direction lateral load reversal." In: *Proceedings of the 9th World Conference on Earthquake Engineering*, VIII, 537-542.
- Kabeyasawa, T., Kato, S. and Hosokawa, Y., (2014). "Effect of spandrel and hanging walls on flexural capacity of reinforced concrete columns with wing walls." In: *10th U.S. National Conference on Earthquake Engineering*, Paper ID 115.
- Orakcal, K., Massone, L. and Wallace, J., (2009). "Shear strength of lightly reinforced wall piers and spandrels." *ACI Structures Journal*, 106(4), 455-465.
- Sanada, Y., Yoon, R., Akahori, T., Ojio, Y., Choi, H. and Kim, Y., (2017). "Softening effects of RC non-structural flat walls on ductile concrete buildings." *Earthquake Engineering and Structural Dynamics*, DOI: 10.1002/eqe.2874.
- SNAP, (2015). "SNAP Ver.7 technical manual." Kozo system. (in Japanese)
- Sugimoto, K., Katsumata, H., Masuda, Y., Nishimura, K., Matsumori, T. and Nishiyama, M., (2017). "Shaking table test of 6-story wall frame building to investigate collapse process of RC buildings." In: *Proceedings of the 16th World Conference on Earthquake Engineering*, Santiago, Paper No. 615.
- The Building Center of Japan (BCJ), (2013). "The building standard law of Japan on CD-ROM." Tokyo: The Building Center of Japan.
- Yanagisawa, M., (2008). "An experimental study on structural performance of R/C frame with non-structural slit wall: part 2 discussion of test results." *Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan*, C-2, Structures IV, 393-394. (in Japanese)