Lateral-force resisting mechanisms of flexure-dominant multi-story structural walls sustained on soft-first-story

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ABSTRACT: This paper presents lateral-force resisting mechanisms of flexure-dominant multi-story structural walls on a soft-first-story. Two reinforced concrete specimens consisted of the 3-story-high structural wall on the soft-first-story were constructed in 1/3-scale and tested under cyclic loading simulating earthquake motions. The test variable was the longitudinal reinforcement ratio in the boundary beam of the 2nd floor, which was 1.19% or 2.56%. The structural walls yielded in flexure as intended, which, however, did not result in the concentration of deformation only on the 2nd story. The reason is that the boundary beam yielded in flexure and tension, and a yielding mechanism other than intended was formed. The difference in load capacity between the two specimens was not of significance. The mechanism needed as large drift angle as approximately 2% to attain the design lateral load capacity. The beam-column joints on the 2nd floor failed, which resulted in an inability to sustain the axial load.

1 INTRODUCTION

To use as stores or parking lots, RC buildings often have an open-first-story whose stiffness is less than other stories. In the past earthquake disasters, collapse or severe damage has been observed in the soft-first-stories of many reinforced concrete buildings which had structural walls in the upper stories. More specifically, the interstory drifts of the soft-first-stories were larger than the other stories and columns failed in flexure or shear. Those cases in 1978 Miyagiken-oki and 1995 Kobe earthquake are well known.

One of the solutions to prevent the collapse or the severe damage is to avoid the concentration of deformation in the first story that is less stiff and weaker than the upper stories. A favorable failure mechanism would be the walls failing in flexure or shear at the second story. Various investigations of multi-story frames with the 2nd-story walls failing in shear have been carried out by Shobu et al. (e.g., 2012) and a design method is being established. On the other hand, research on the frames with the 2nd-story walls failing in flexure is insufficient for proposing a design procedure. It is assumed that boundary beams, on which flexure-dominant multi-story structural walls are built, should be subjected to large moment and tensile force induced when the upper walls reach their flexural strength. There are some suggestions but no clear rule for designing boundary beams is available in design regulations or in AIJ Standard for Structural Calculation of Reinforced Concrete Structures (2010). Boundary beams currently designed might not be strong enough to resist such moment, which leads to a collapse mechanism different from the designed. To investigate the issue above, the authors constructed two 3-story-high structural walls on the soft-first-story and conducted cyclic loading tests simulating seismic loads.

2 TEST DETAILS

2.1 Specimens

Figure 1 shows the configuration and reinforcement arrangement of the two specimens (P1, P2). Table 1 lists reinforcement details. 1/3-scale two specimens were constructed. To ensure flexural yielding of the wall, the cross-sectional area of the boundary columns in the 2nd story and above was smaller than the 1st story columns. Accordingly some longitudinal reinforcing bars were anchored in the beam-column joints at the 2nd floor using 180-degree hook. For the same reason, some additional longitudinal reinforcement was provided at the bottom of the 1st story columns. They had enough development length and cut-off length as shown in Figure 2. Table 2 summarizes mechanical properties of the materials. The design story shear force of the 1st story was about 1.2 times larger than the 1st story shear force induced by the failure mechanism in which the 2nd story wall was assumed to be failed in flexure. The design story shear forces were calculated by using material properties shown in Table 2.

The test variable was the longitudinal reinforcement ratio in the boundary beam of the 2nd floor. As the wall deformation progressed, much larger bending moment was imposed on the boundary beam. Sakashita et al. (2010) proposed assumptions for calculating the moment that can act on the foundation beam constructed under flexure-dominant walls. Following the procedures, the authors conducted a simple frame analysis as shown in Figure 3. The strength required for the boundary beam was roughly calculated by the analysis which modeled the soft-first-story and considered forces transmitted from

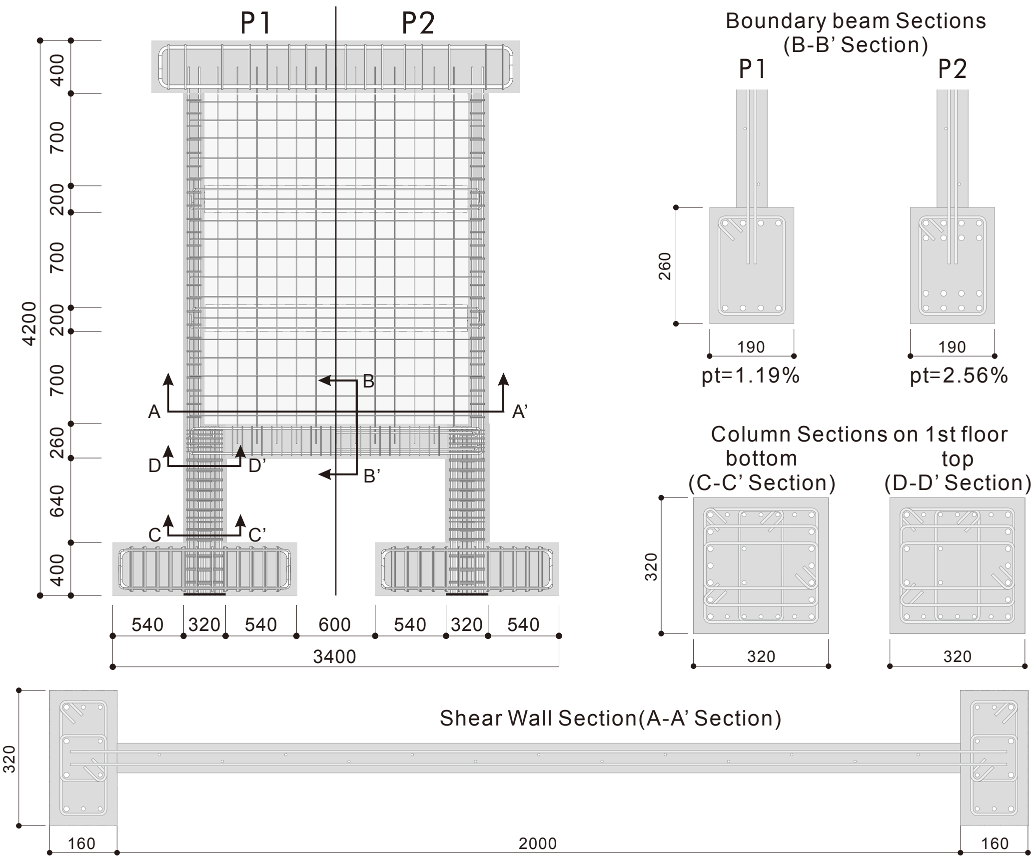


Figure 1. Configuration and reinforcement arrangement (unit: mm)

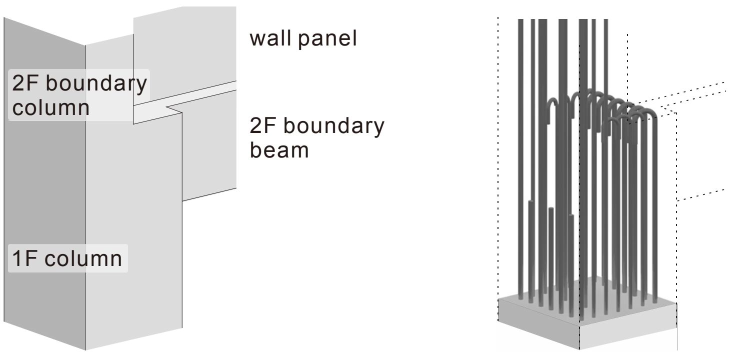


Figure 2. Detail of column longitudinal reinforcement inside beam-column joint

the wall. Figure 3 (a) shows the details of the analysis model. In this analysis, material properties were obtained by the material tests. The columns in the soft-first-story were idealized in a fiber model. The stress-strain curve of the reinforcement was modeled by a bi-linear model and the stiffness after yielding was 0.01 E_s. E_s is Young's modulus. Concrete was modeled as a quadratic curve to the peak point and the peak stress was kept beyond the peak point. The boundary beam was modeled as an elastic beam, which resulted in GA, EA and EI were constant. The beam-column joints were assumed to be rigid. Applied forces, N_c, N_t, T_w, Q and M_c are shown in Figure 3(a).

where N_c =axial compression force acting on the boundary column (=N_t+T_w+N_L); N_t= tensile yielding strength of the boundary column; T_w = total tensile force assuming that all the vertical reinforcement in the wall panel yields; N_L = constant axial force; Q= lateral force; and M_c = flexural strength of the boundary column under the axial compression force of N_c.

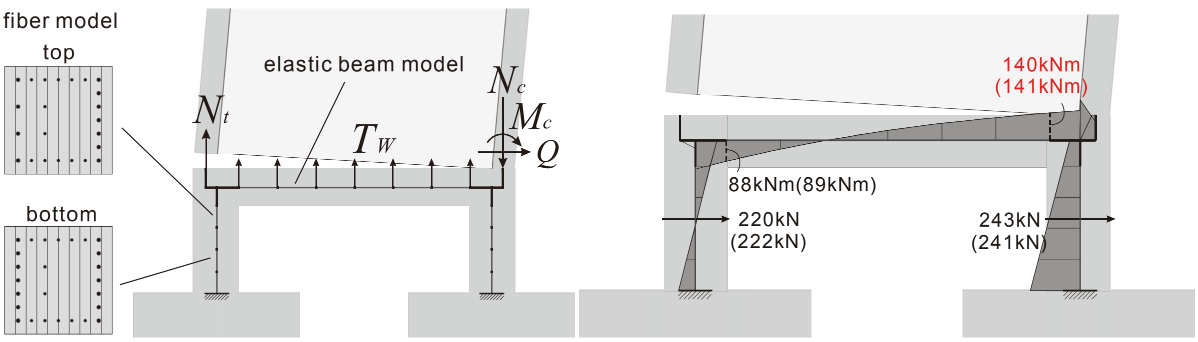
Table 1. Reinforcement details

Member (Section : mm)	Bar Type			Steel Ratio
1F Column (320 x 320)	Longitudinal	Bottom	14-D13, 12-D10	2.57%
		Top	11-D13, 12-D10	2.20%
	Transverse	In plane	6-D6@75	0.79%
		Out of plane	4-D6@75	0.53%
2-4F Column (320 x 160)	Longitudinal		4-D13, 6-D10	1.83%
	Transverse	In plane	4-D6@100	0.40%
		Out of plane	2-D6@100	0.40%
Wall Panel (Thickness: 70)	Vertical, Horizontal		D6@150	0.30%
2F Beam (190 x 260)	Longitudinal	P1 Top, Bottom	4-D13	1.19%
		P2 Top, Bottom	8-D13	2.56%
	Transverse		2-D6@75	0.44%
3-4F Beam (70 x 200)	Longitudinal	Top, Bottom	1-D10	0.58%

Table 2. Mechanical properties of materials

Concrete			
Specimen	Compressive Strength, F_c (MPa)	Young's Modulus (GPa)	Tensile Strength (MPa)
P1	34.9	27.9	2.54
P2	33.3	27.0	2.18

Reinforcement			
Type	Yield Strength (MPa)	Young's Modulus (GPa)	Tensile Strength (MPa)
D6 (SD295A)	441	181	530
D10 (SD295A)	377	186	532
D13 (SD345)	379	183	553



(a) Analysis model (b) B.M.D. calculated by using P1's (or P2's) material property

Figure 3. Strength required for boundary beam

Figure 3 (b) shows analytical results. The maximum bending moment and the axial tensile force imposed on the elastic boundary beam is about 140kNm and 220kN, respectively. The bending moment is too large to be sustained by boundary beams even if the much amount of longitudinal reinforcement is arranged. The longitudinal reinforcement ratios in the boundary beam of the 2nd floor were determined to be 1.19% in P1 or 2.56% in P2. The ratio of 1.19% in P1 is common in Japanese RC buildings. The ratio of 2.56% in P2 is about twice as much as that of P1. The flexural strengths under the axial tensile force calculated by this analysis are 15kNm in P1 and 43kNm in P2, which are much smaller than the maximum bending moment (140kNm). The tensile yielding strengths of the boundary beams are 384kN in P1 and 768kN in P2, which are larger than the axial tensile force calculated by the analysis (220kN).

2.2 Loading Setup and Measurement

Figure 4 shows the loading setup. Lateral load was applied statically through the 2000kN hydraulic jack. Two 1000kN hydraulic jacks applied axial force of 850kN, which corresponded to $0.1A_gF_c$ on the structural wall, and adjusted the point of contra flexure to 5000mm above the top surface of the base block. A_g is a gross sectional area of the columns and wall, and F_c is the compressive strength of concrete. The load of each vertical hydraulic jack depends on the lateral load Q to be applied as

$$N_W = -0.7Q + 425(\text{kN}) \;\;,\; N_E = +0.7Q + 425(\text{kN}) \qquad (1)$$

The drift was defined as the horizontal displacement of the center point of the 4th floor beam at 2600mm above the top surface of the base block. The horizontal displacement was measured by displacement transducers attached on the front side of the specimens. The drift angle, R was obtained by dividing the drift by 2600mm. The first loading cycles to 0.05% was followed by two full cycles to each of the drift angle of 0.1%, 0.25%, 0.50%, 0.75%, 1.0%, 1.5% and 2.0%. Displacement transducers were attached to measure the deformation of each member, and strain gages were attached on reinforcement. Four 3-axis load cells were installed under the base blocks to obtain the axial force, N, shear force, Q and moment, M acting on the 1st story columns.

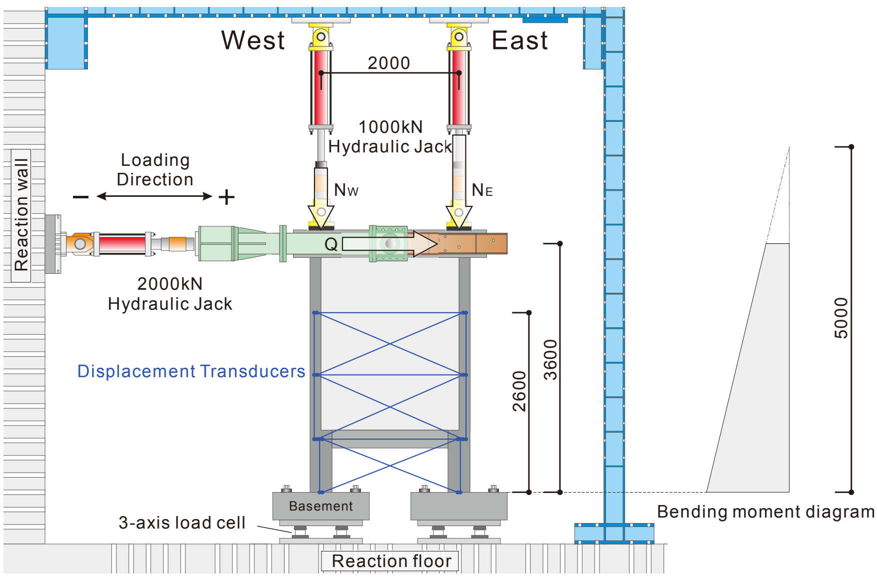


Figure 4. Loading setup (unit: mm)

3 TEST RESULTS

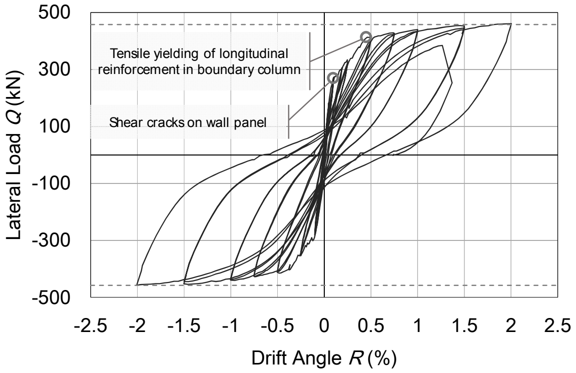
3.1 Lateral load – Drift angle relations and crack distribution

Figure 5 shows lateral load - drift angle (Q-R) relations. The dashed lines show the design flexural load capacity of the wall that is calculated using a design equation of AIJ Design Guidelines for

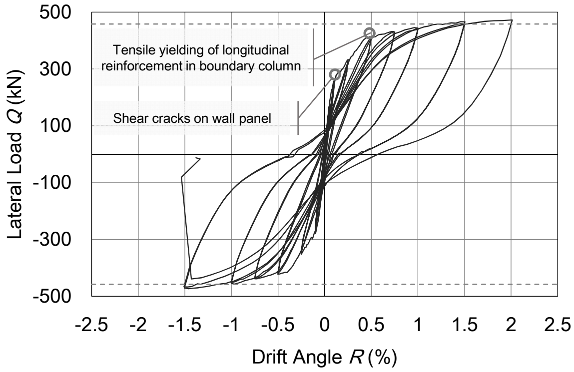
Earthquake Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept (1999). The equation is as follows,

$$Q_{mu} = {}_{w}M_y/h = \left(a_t \sigma_y l_w + 0.5\sum\left(a_w \sigma_{wy}\right) l_w + 0.5 N l_w\right)/h \qquad (2)$$

where h = the shear span length of the wall measured from the top surface of the boundary beam on the 2nd floor; a_t = the total cross sectional area of longitudinal reinforcement in the boundary column subjected to tensile axial load; σ_y = the yield strength of longitudinal reinforcement in the 2nd story boundary column; l_w = the length between the centers of the boundary columns in the 2nd story; a_w = the cross sectional area of vertical reinforcement in the wall panel; σ_{wy} = the yield strength of vertical reinforcement in the wall panel; and N = axial force.



P1



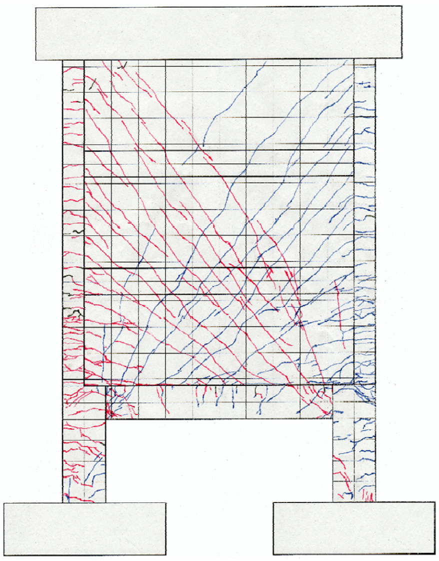
P2

Figure 5. Lateral load – drift angle relations

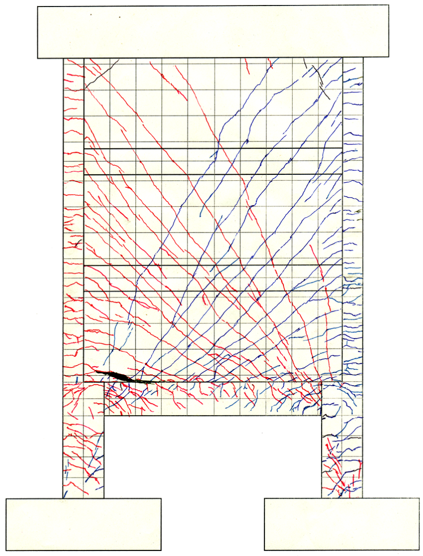
Crack distribution at R=+2.0% is shown in Figure 6. Because the behaviors in the positive and negative directions of loading are almost the same, the one in the positive loading will be mainly discussed. Both specimens showed the initial shear cracks in the wall panels and flexural cracks in the tensile columns in the 1st story at R=0.1%. At R=0.25%, the initial flexural cracks in the boundary columns in the 2nd story were observed and some of them developed into the wall panels and the boundary beams as flexure-shear cracks. All the longitudinal reinforcement in the boundary columns in the 2nd story yielded in tension at R=0.48% in P1 and R=0.46% in P2, which is defined as flexural yielding of the walls. At deformation larger than R=1.0%, the number of flexure-shear cracks in the walls did not increase and the width of a few of the cracks kept expanding. The top longitudinal reinforcement in the boundary beams of the 2nd floor yielded in tension at R=0.68% in P1 and R=0.76% in P2. At R=1.5%, vertical cracks were observed in the extreme compression fiber of the compressed columns in the 1st story and the cover concrete slightly spalled off in P1. Moreover, at about R=1.5% in P1 and about R=2.0% in P2, the measured tensile strain on the reinforcement at the bottom of compressed columns exceeded the yield strain obtained by the material test. It is considered that the columns yielded in flexure at about R=1.0% in view of the transition of the strain gages' value and the bending moment measured by 3-axis load cells.

The maximum lateral loads, which were 460.9kN in P1 and 471.9kN in P2, were obtained at R=2.0%. Both specimens attained the design flexural load capacity of the wall of 458kN at the drift angle of approximately 2%, which was larger than expected in the design. Concrete crushing was observed at the bottom of the wall panels on the 2nd floor, while no damage was observed in the compressed boundary columns. Damage and cracks observed in the specimens suggest that a compression strut connecting the loading point at the top of the wall and the bottom of the 1st story column under compression should be formed.

Ultimately, the beam-column joint on the 2nd floor failed as shown in Figure 7 and the specimens were not able to sustain lateral and axial loads. Just before the failure, the separation between the wall panels and the compressed boundary columns was observed and diagonal cracks formed in the beam-column joints. Lateral and axial loads transferred through the compressed boundary column on the 2nd floor as shear and axial forces were assumed to be increased by the damage at the bottom of the wall panels. Top longitudinal reinforcement in the boundary beams of the 2^{nd} floor, longitudinal reinforcement in the boundary columns and transverse reinforcement in the beam-column joints were arranged on the trajectory of the diagonal cracks. Both specimens failed along the diagonal cracks.

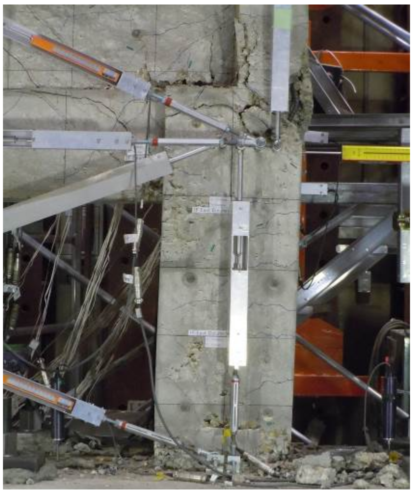


P1



P2

Figure 6. Crack distribution at R=+2.0%



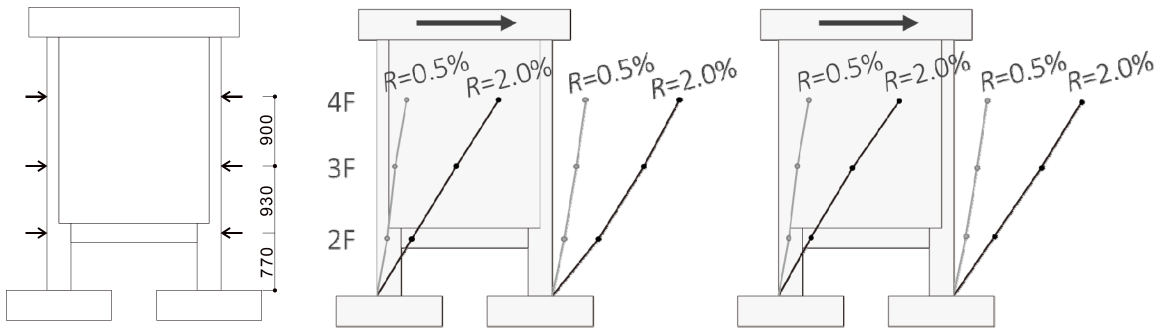
P1

P2

Figure 7. Damage at failure

3.2 Horizontal displacement of floors

Figure 8 shows horizontal displacements at the locations indicated by arrows measured by the displacement transducers. If a lateral force resisting mechanism in which the wall yielded at the bottom had been formed, it would have led to much smaller interstory drift in the 1st story than the upper stories. However, the interstory drift of the 1^{st} story continued to increase even after the structural walls yielded in flexure at about R=0.5%. The displacement distribution along the height of the specimens is approximately linear. These results revealed that a mechanism different from the one in which the wall yielded in flexure and therefore the open-first-story was protected was observed.



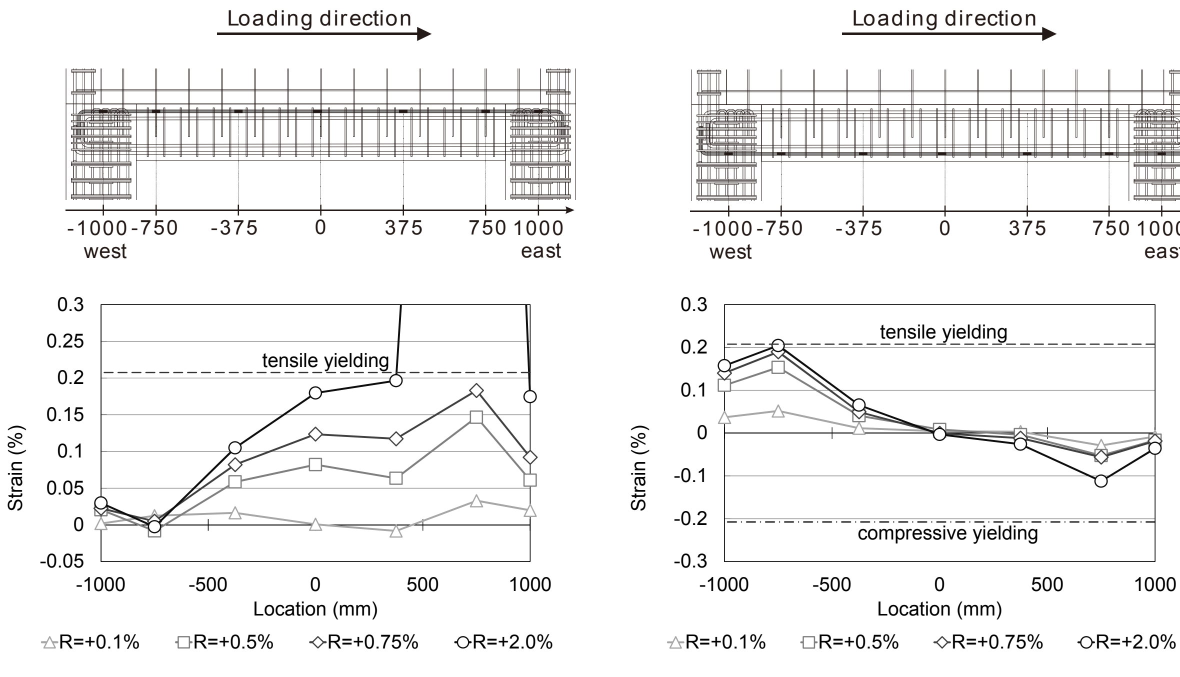
Location of displacement transducers P1 P2

Figure 8. Horizontal displacements at floors (magnified by 30, unit: mm)

3.3 Deformation of the boundary beams and lateral-force resisting mechanism

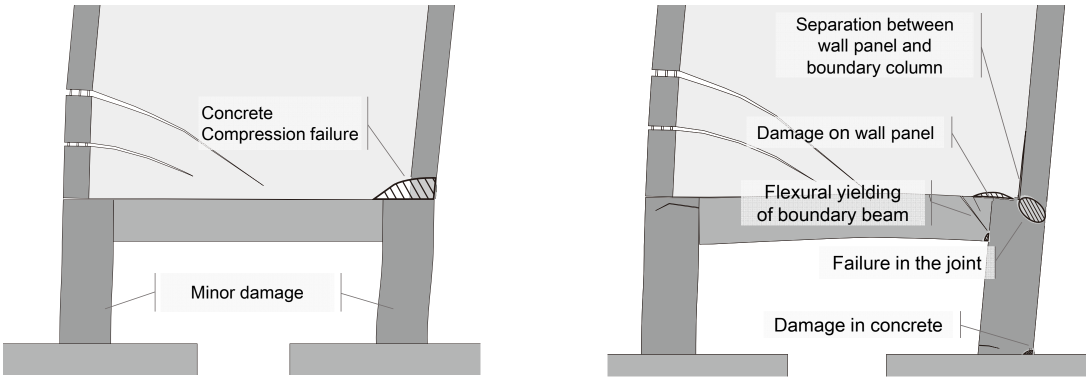
Figure 9 shows strains measured in the longitudinal reinforcement in the boundary beams of the 2nd floor. Some flexure-shear cracks developed from the wall to the boundary beams. Tensile reinforcement roughly yielded at the end of the beam. Yielding of the top longitudinal reinforcement was observed at *R*=0.68% in P1 and *R*=0.76% in P2.

Failure mechanisms intended in the design and observed in the tests are illustrated in Figure 9. If the boundary beams had been rigid and strong enough to resist the moment from the bottom of the wall and the top of the 1st story columns, deformation and damage should have concentrated in the wall. The resisting mechanism developed in the tests imposed large deformation on the bottom of the 1st story column accompanying the large deformation of the boundary beam although the boundary beam was assumed to be stiff in practical design. Not only the 2nd story boundary column but also the 1st story column and the 2nd story beam-column joints determined the deformation capacity of this structural system.



The top longitudinal reinforcement (P2) The bottom longitudinal reinforcement (P2)

Figure 9. Strain of longitudinal reinforcement in boundary beams



Intended in the design — Experimental results

Figure 10. Lateral-force resisting mechanisms

4 CONCLUSIONS

Two reinforced concrete specimens consisted of the 3-story flexure-dominant wall on the soft-first-story were constructed in 1/3-scale and tested under cyclic loading to simulate earthquake motions. The experimental parameter was the longitudinal reinforcement ratio in the boundary beam of the 2nd floor.

- The input bending moment of the boundary beams constructed under flexure-dominant walls was calculated by frame analysis. The ratio of flexural strength of the boundary beam to the maximum input moment in the analysis was 11% in P1 and 31% in P2. Yielding of the longitudinal reinforcement of the boundary beams was observed in the test.
- Both specimens attained the design load capacity at R=2.0%. At the same time, concrete crushing was observed at the bottom of the wall panels on the 2nd floor, while there was almost no damage observed in the boundary columns in compression. Ultimately, the beam-column joint on the 2nd floor failed and the specimens were not able to sustain the axial load.
- The lateral-force resisting mechanism formed after tensile yielding of longitudinal reinforcement in the boundary columns on the 2nd story. This was different from the one that flexure-dominant structural walls built on the rigid boundary beam. Not only the 2nd story boundary column but also the 1st story column and the 2nd story beam-column joints determined the deformation capacity of this structural system.

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