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FULL-SCALE STATIC LOADING TEST ON A FIVE STORY REINFORCED CONCRETE BUILDING (PART1: OUTLINE OF THE TEST)

Toshikazu Kabeyasawa(1), Tomohisa Mukai(2), Hiroshi Fukuyama(3), Haruhiko Suwada(4) and Hiroto Kato(5)

(1) *Associate Professor, Tokyo Metropolitan University, tosikazu@tmu.ac.jp*
(2) *Senior Research Engineer, Building Research Institute, t_mukai@kenken.go.jp*
(3) *Director, National Institute for Land and Infrastructure Management, fukuyama-h92ta@nilim.go.jp*
(4) *Senior Researcher, National Institute for Land and Infrastructure Management, suwada-h92h9@nilim.go.jp*
(5) *Senior Research Engineer, Building Research Institute, pckato@kenken.go.jp*

Abstract

The paper shows the static loading test results on full-scale reinforced concrete building utilizing columns with wing walls. The specimen is a five-story reinforced concrete building with 1×2 bays and height of about 19m. The wall frame has large openings in longitudinal direction. The gaps was formed along the edge of wing walls to make a frame consisting of columns with wing wall and beams. Attached wing walls works as a rigid zones of soften beams and additional column section, and the specimen shows calculated strength of beam side sway mechanism. The moment curvature distribution of the 1st story wing wall frames are evaluated experimentally in the test. The inflection point derived from the interpolation between moment resisting frame and wall frames approximates the test results. The residual crack width of the beam elements becomes large for the damage concentration on the local hinge region. Two types of windows are installed in openings on 1st floor. The concrete mortar and fibrous material are used in each for fixing the windows. The flexible fixing method improved the capacity, but both windows is not available with story drift 0.80%.

Keywords: Reinforced concrete structure, Full scale test, Gap, Columns with wing wall, Damage control design

1. Introduction

2011 Great East Japan Earthquake is a large-scale subduction zone earthquake, and a middle-scale earthquake ground motion was observed in widespread North East area of Japan. A number of reinforced concrete buildings suffered damages such as concrete cracking [1]. As mentioned in post-earthquake damage observation, a several reinforced concrete buildings designed according to the current Japanese seismic code, was not operational after the earthquake, although those did not show critical damages of the building such as pancake collapse or story collapse. The current Japanese seismic design code requires only two items; 1) the stress on the materials (steel rebar or concrete) of the building does not exceed the allowable stress in a frequent ground motion, 2) the building does not collapse and confirm the safety of human lives in extreme ground motion. It allows the loss of post-earthquake functionality of the building in a large-scale earthquake. However, the seismic performance of the building required by Japanese society has been changed through the experience of recent severe earthquakes in Japan. In addition to the human lives assured by the code, the building owners hope for the design of post-earthquake functionality after large-scale earthquake. So far, the importance factor for the public buildings, which is originally a safety margin against extreme ground motion, was substituted as the index for the post-earthquake functionality [2]. In order to attain the target perform that the building is operational without any repair or a large-scale repair after a large-scale earthquake, the design requirement must be refined through the post-earthquake damage observation and full-scale test of the buildings. In this study, the new construction method within a conventional structural design framework is proposed, which contributed on the damage control design, and different from the newly technology such as damper system or base isolation system.

2. Proposed damage control design

In proposed design, it utilize the wing wall, which is regarded as the nonstructural panels in conventional design. The wing wall is designed as a part of the attached column section, and the total frame obtains base shear coefficient higher than 0.40 by shortening the clear span length of the beams. The damage of the building under a large-scale earthquake remains minor due to this high seismic performance. The end of the concrete wing wall section has sufficient width and confinement by the hoops, in order to prevent the compression failure in small drift ratio. The other panels such as hanging walls, spandrel walls and mullion walls are separated from the main frame by forming gaps from the surrounding beam and columns, it reduced damage level of concrete cracking, realize obvious beam side sway mechanism in structural design.

With this design concept, the maximum story drift reduce and the residual hysteretic energy capacity remains under a large-scale ground motion compared to the normal moment resisting frame by giving high stiffness and strength on the frame. It also prevents disability due to the damage of facility equipment and non-structural elements. The location of the plastic hinge on beams moves to the end of the wing wall section, so that it prevents the beam-column joint failure, which is difficult to repair after the earthquake. On the other hand, the clear span of the beam is short, and the rotation angle or ductility factor of the beams turn to be high compared to the normal moment resisting frame with same story drift. The shear stress on the beam is also higher.

3. Specimen

3.1 Outline of the Specimen

To verify the proposed damage control design, the static loading test on full-scale reinforced concrete building is carried out [3]. The plan and elevation of the test specimen is shown in Fig.1. The specimen is a full-scale five story reinforced concrete building with 2 span and 1 bay. The specimen was built in the laboratory of Building Research Institute at Tsukuba, Japan. The frame has the infilled concrete wall in longitudinal section. The story height is 3.5 m. The total height of the building is 18.7 m. The span length is 6 m both in longitudinal and transvers directions. The total mass of the super structure is about 550 tons. The base foundation of the specimen is fixed on the reaction concrete slab by PC reinforcement rods. Two types of the opening (2.0× 1.8m, and 1.0m ×1.8m) are symmetrically provided on the walls in Y0 and Y1 section. The perimeter ratio of openings is 0.51.

The walls are divided into wing wall, spandrel, and hanging walls by these openings. In typical Japanese middle-high reinforced concrete building, it forms a gap between main frame and partition walls in order to make an obvious beam side sway mechanism. In this specimen, the gap is provided at the end of the openings in vertical direction, so that the frame is consist of beams and columns with wing wall. The mullion walls between openings are completely separated from the main frame. The width of the gap is 45mm at the end of the wing wall, and 80 mm at the end of the mullion wall, which is wider than usual to prevent the conflict of the concrete wall.

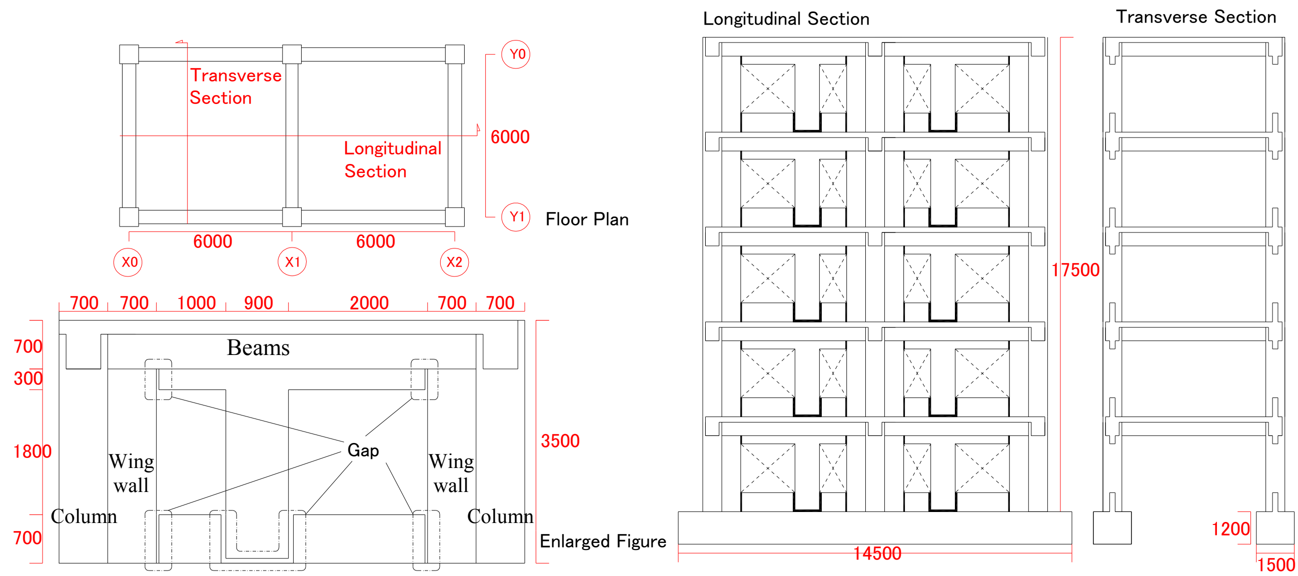


Fig. 1 – The full-scale specimen (unit mm)

3.2 Section and Reinforcement of Members

Fig.2. shows section of the beam and column members. Because both Y0 and Y1 frame are outer frames, the axial load on those columns is smaller than that on the columns inside as well as the area of supporting floor section. In this test, the column section is designed under twice weight of the structure in order to represent the standard section in the multi-span building in transvers direction. The section and reinforcement of the beam and columns are determined by two conditions as well as the conventional structural design [4]; 1) the maximum story drift does not exceed 0.5% under seismic design load (C0 = 0.2), 2) the base shear coefficient exceed 0.3 as a lateral load carrying capacity for the plain moment resisting frame without wing walls. This is because the frame is usually designed as a moment resisting frame by separating infilled concrete walls from main frames in conventional design. Columns are 700 mm square section with sixteen D25 rebar (It indicates a diameter of rippled steel rebar is about 25 mm) as longitudinal reinforcing rebar. The hoop is double D13 rebar at 100 mm interval (1st and 2nd story) and D13 rebar at 100 mm interval (from 3rd to 5th story). Beams are 500×700 mm section with eight D25 rebar (from 2nd to 4th floor), and with six D25 rebar (5th floor and top beam) as longitudinal reinforcing rebar. Stirrup is D13 rebar at 100 mm interval. Steel type of longitudinal reinforcing rebar upgrade to SD390 (It indicates nominal yield strength of the steel rebar is 390 N/mm^2) for columns on upper story. Steel type of rebar is SD295, which diameter is smaller than 13 mm.

Fig.3. shows the detail section of the columns with wing walls. Wing wall section attached to the columns in X0, X1, and X2 frame has identical reinforcement. The length is 700 mm, and width is 200 mm. For wing wall section, a ratio of the wall thickness to the column depth is high and bar arrangement is double layer in order to prevent compression failure at the end of the concrete section in small story drift. The length of the wing wall is determined so as to base shear coefficient exceeded 0.40 for beam side sway mechanism with plastic hinges on the end of beams and bottom of the columns with wing walls. The end of the wing wall section is specially reinforced by six D16 longitudinal rebar and confined by hoops to prevent the buckling of the longitudinal rebar. The longitudinal reinforcement in the wall section is D10 rebar with double layer at 200 mm

intervals, which is confined by the spreader bar (D10). The transverse reinforcement (D10 rebar) in the wall section is anchored in the column section, and by 180 degree hook in the wing wall section. The intervals of those transverse reinforcement is 100 mm for 1st story, and 200 mm for other stories. This is also special reinforcement detail to prevent the bucking of the longitudinal reinforcement. It confirms the flexural failure precedes the shear failure for 1st story columns with wing walls by the calculation.

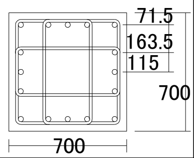
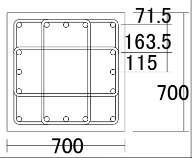
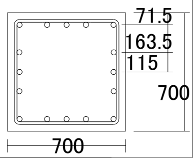
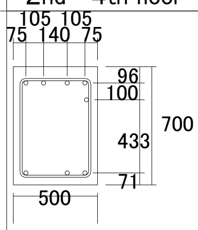
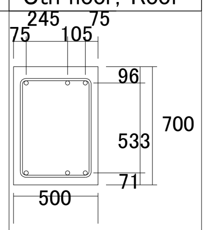
Columns	1st story	2nd story	3rd～5th story
Section			
Main bar	16D25 (SD345)	16D25 (SD390)	16D25 (SD390)
Hoops	4D13 (SD295A)	4D13 (SD295A)	2D13 (SD295A)
Hoops in Beam Column Joint		2D13 (SD295A)	2D13 (SD295A)
Beams	2nd～4th floor	5th floor, Roof	
Section			
Main bar	8D25 (SD345)	6D25 (SD345)	
Stirups	2D13 (SD295A)	2D13 (SD295A)	

Fig. 2 – Section and reinforcement of members (unit mm)

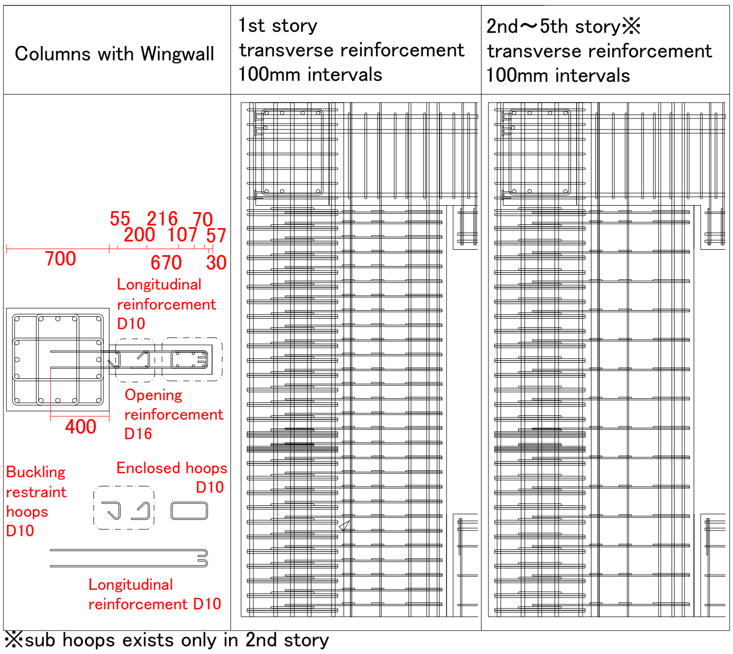


Fig. 3 – Section and reinforcement of columns with wing walls (unit mm)

Fig.4. shows the reinforcement of the floor slab. The slab thickness is 200 mm. The top and bottom of the reinforcement are basically D10 rebar at 150 mm intervals, but D13 rebar alternates with D10 rebar for top reinforcement in transverse direction. The lap splice of the reinforcement is provided in middle of the span, and the end of top reinforcement is anchored to the transverse beams by 90 degree hook. The anchorage length of the bottom reinforcement is 250 mm from side surface of the transverse beam.

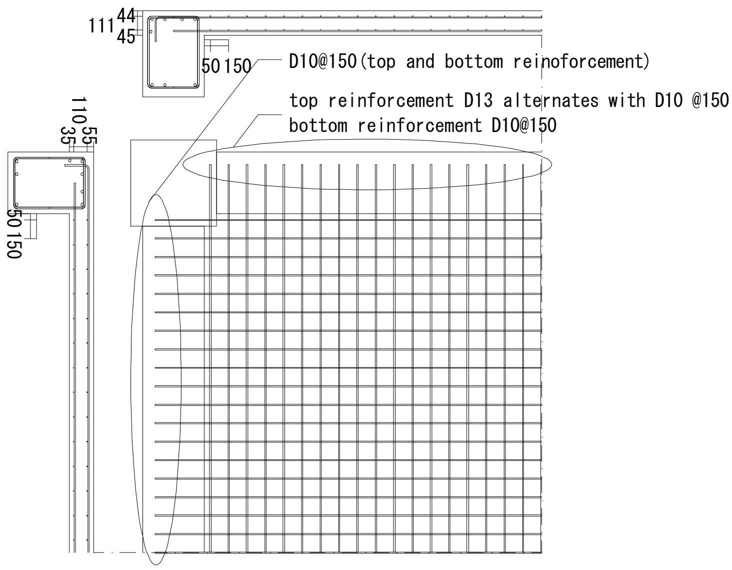


Fig. 4 – Section and reinforcement of floor slabs (unit mm)

Table 1 shows the material test result of concrete and rebar. The design concrete strength is 30 N/mm^2 in the test. The material test of the concrete has been carried out on first day of the loading test. Concrete casting has been conducted on August 21th for 1st story, September 10th for 2nd story, September 29th for 3rd story, October 17th for 4th story, and November 5th for 5th story. The static loading test has been conducted from December 16th to January 18th.

Table 1 –Material test results

Steel bar (N/mm^2)	Wall reinforcement Slab reinforcement	Enclosed Hoops	Longitudinal bar of columns and beams	Hoops of columns Stirup of beams	Longitudinal bar of columns	Opening reinoforcement
	D10 (SD295A)	D10 (SD295A)	D25 (SD345)	D13 (SD295A)	D25 (SD390)	D16 (SD345)
Yield strength	352	372	383	340	449	384
Tensile strength	482	550	568	498	628	552
Concrete (N/mm^2)	Start date of the loading test (12/16)					
	Foundation	1st story 2nd floor	2nd story 3rd floor	3rd story 4th floor	4th story 5th floor	5th story Roof
Conpressive strength	38.6	34.9	33.0	37.7	33.6	31.3
Young modulus	2.92×10^4	2.86×10^4	2.61×10^4	2.85×10^4	2.62×10^4	2.47×10^4
split-tearing tensile strength	2.76	2.71	2.58	2.68	2.35	2.61

3.3 Loading Set Up

Fig.5. shows the attachment of the actuators on the top floor. Eight actuators are used in the static loading test, and a series of 4 actuators located on the roof level and 4th floor level in each. Those actuators induced lateral force on two concrete blocks settled in top and bottom of the transverse beams in X1 frame through the steel pipes. They pinched the center of the floor slab from upper and lower level. The other side of the actuators are connected to the huge reaction wall. The moment center height of the lateral load is equalized with the center of the floor slab width. The maximum capacity of the actuator is ±1000 kN within ±500 mm. Two actuators on the top of the specimen is controlled by the displacement of the specimen from the reaction wall. Other actuators follows load of those two actuators.

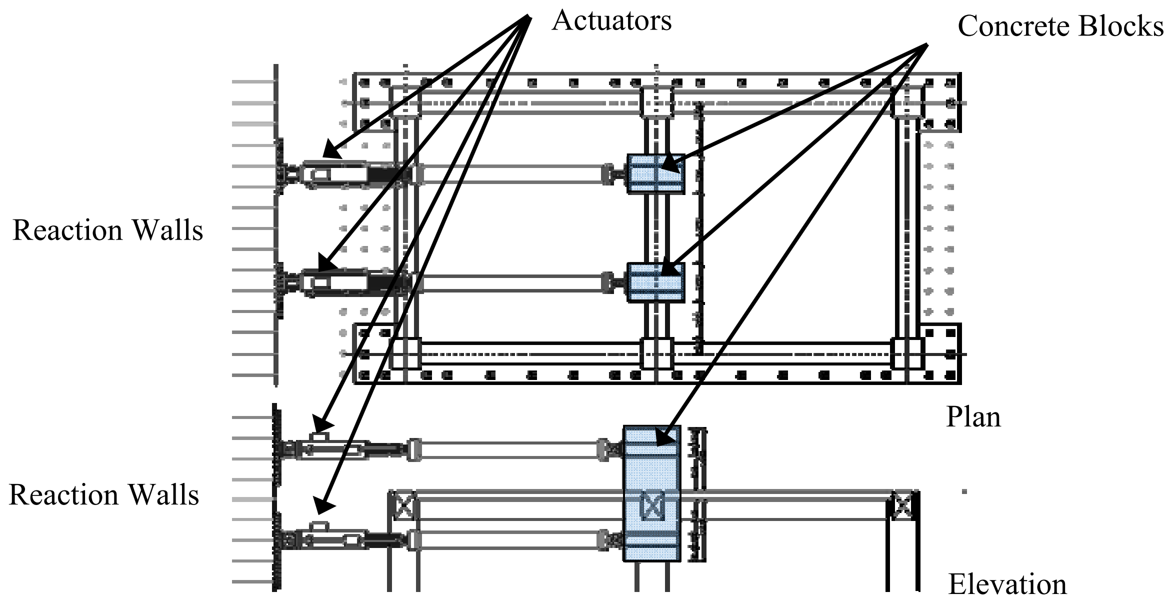


Fig. 5 – Loading set up on the roof level (similar detail in 4th floor)

The sum of the lateral load on 4th floor level is twice of that on roof level, where a ratio of the overturning moment to the base shear is identical with that under the inverted triangle seismic force. It confirms the similar load-displacement relationship on 1st, 2nd and 3rd story are obtained with those two different loading pattern by frame analysis before the test. The peak displacement is defined by total drift (the drift ratio of displacement on the roof level to the total building height). The tensile force is induced at first in the test. One cycle loading has been conducted for 0.065% and 0.125%, and two cyclic loading has been conducted for 0.25%, 0.50%, 1.00%, 1.50% and 2.00% drift. Absolute floor displacement and member drift ratio is measured by displacement meters. The floor displacement is measured from the reaction wall (North) and the steel braced frame (South) considering elongation of the beams. The displacement of the floor is expressed by the mean value of those two record. The local displacement such as moment curvature and shear deformation is specially measured for 1st story column and 2nd floor beams. The strain gauges are attached on the rebar of column, beam, wing wall, and floor slab section. The position of the concrete crack and the maximum crack width is recorded in peak and unloading state.

4. Result of the full scale test

4.1 Load displacement relation

Fig.6. shows load displacement relation between the total drift and base shear. Table 2 shows the maximum story drift (3rd story) and base shear in loading peak. Increment of the restoring force after 1.0% drift is small due to

forming the plastic hinge mechanism in each member. The columns with wing wall normally shows strength deterioration after compression failure of end section, but this frame shows ductile behavior until ultimate state because the contribution of the beam members are dominant. The strength and stiffness increase locally due to the conflict of the gap between mullion walls and spandrels after 1.5% drift. The inverted S-shape slip behavior is appeared in 2nd cyclic loading. The collapsing load of beam side sway mechanism based on the flexural moment strength of each member is 4654 kN in calculation, which is consistent with the maximum strength (4489 kN) before the conflict of walls. Fig.7. shows the distribution of the story drift at loading peak. The maximum story drift was obtained at 3rd story except for 0.125 % drift in tensile direction. The story drifts of 2nd and 3rd floor are larger than the total drift, and the member damage concentrated on those stories. The story drifts of 4th and 5th floor are quite smaller than other stories. This is because the story shear change apparently in this section due to the reduced loading point in the test.

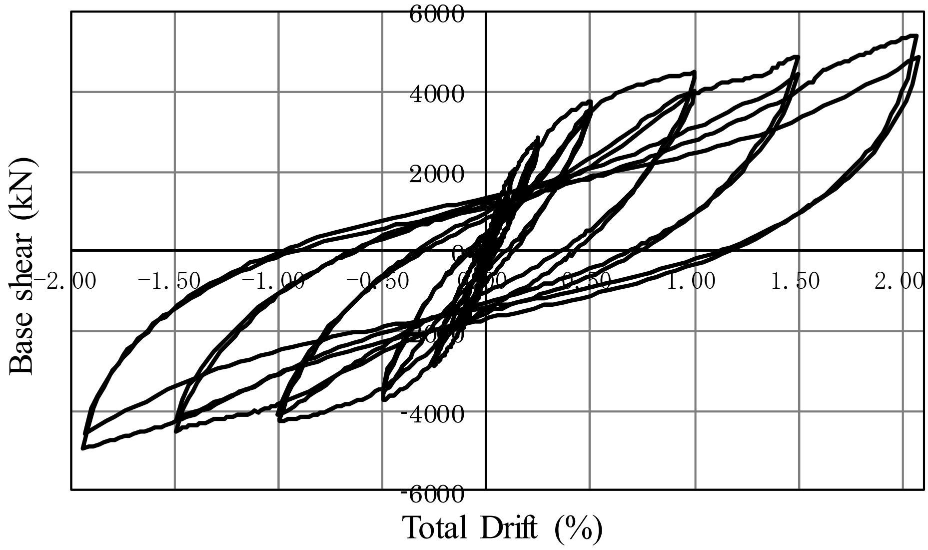


Fig. 6 – Load-displacement relation

Table 2 –Peak response of the specimen in each loading cycle

Total drift	Base shear (kN)		Drift ratio of 3rd story (%)	
Direction	Positive	Negative	Positive	Negative
0.125 %	1979	1976	0.18	0.19
0.25 %	2835	2850	0.35	0.35
0.50 %	3781	3729	0.70	0.73
1.00 %	4489	4211	1.35	1.38
1.50 %	4888	4510	1.90	1.96
2.00 %	5413	4935	2.46	2.50

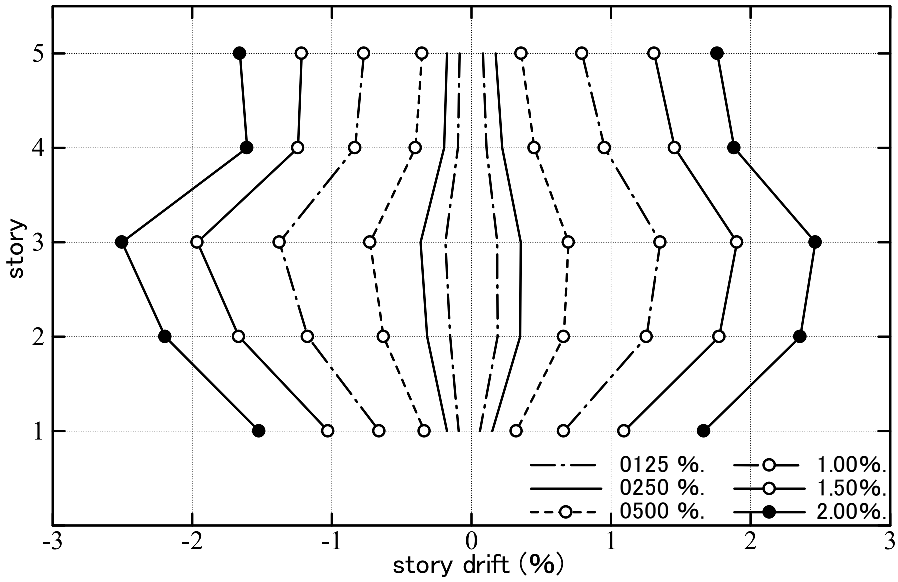
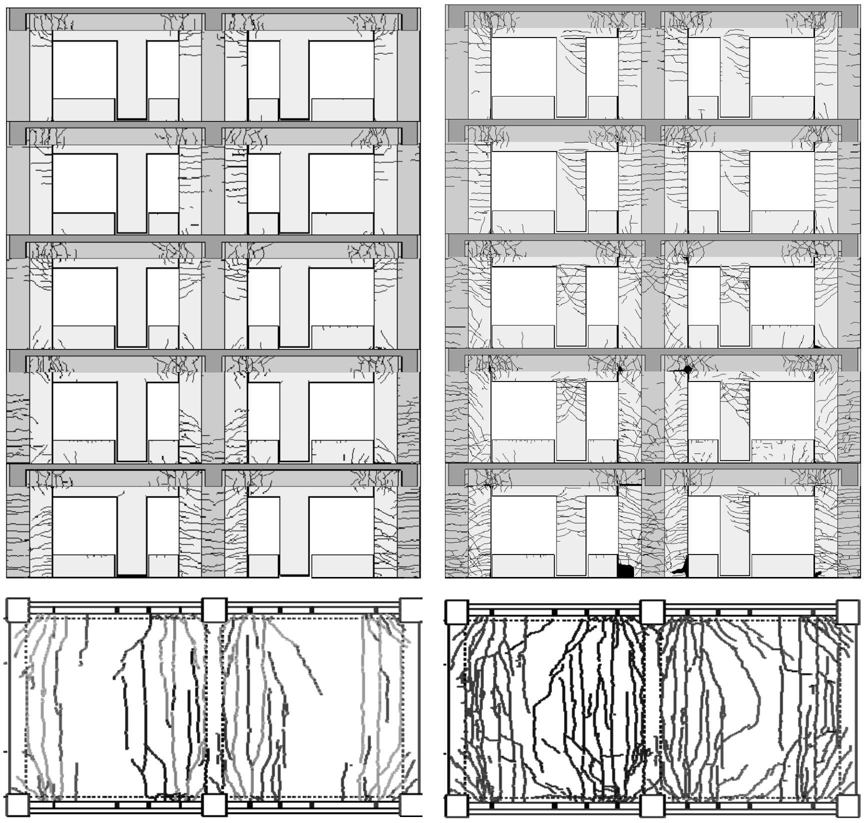


Fig. 7 – Distribution of the story drift

4.2 Damage Pattern

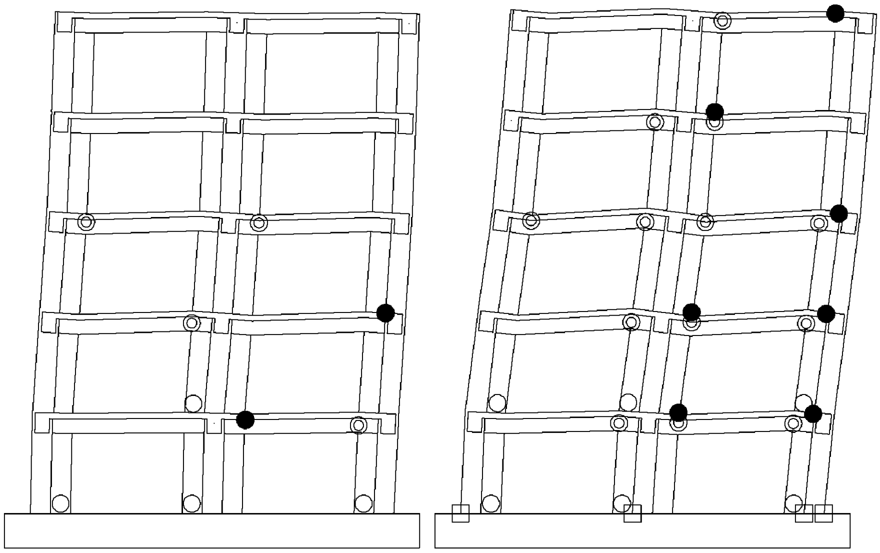
Fig.8 shows cracking patterns of the specimen at 0.5% drift and ultimate state. Representative cracking damage at each peak drift is as follows. The flexural cracking on beam and floor slab is observed around the gap, and the flexural crack is also observed in 1st story columns at 0.125% drift. The flexural crack is observed at the bottom of 2nd story column, and at the top of the 3rd, 4th 5th story columns at 0.25%. At 0.50% drift, the flexural cracks on beams develop and the width of those cracks increased, but the number of the cracks does not increase so much from 0.25% drift. The compression failure of wing wall concrete section is observed in X1 frame at 1.0 % drift. The compression failure for beam members, and the buckling of the longitudinal reinforcement for columns with wing wall are observed at 1.5% drift. Also the bending shear cracks occurs by the conflict of mullion walls to the spandrels in 1st, 2nd, 3rd story at 1.5% drift. The same bending shear crack occurs in 4th and 5th story at 2.0% drift. The flexural cracks on beams are observed around the gap, and it does not develop to the beam column joint finally. This gap helps the frame makes the beam side sway mechanism. The crack on the floor slab is parallel to the transvers beam. The cracks on the columns with wing wall concentrated on the bottom of the columns in 1st and 2nd story, so that it indicates the inflection points of the member is relatively higher for the attached wing walls. On the other hand, the cracks are observed at the top of the columns in 3rd, 4th, 5th story, and it indicates the inflection points is low. The gap width around mullion walls is designed not to conflict to the spandrels until 3.0% story drift, but they conflict due to the flexural deformation for the beam members. Table 3 shows the maximum residual crack width for unloading from 0.25%, 0.50%, and 1.00% drift. The residual crack width shows higher value in beam members except for columns of X1 frame in 1st story. The maximum crack width in the frame recorded at beams in 2nd, 3rd and 4th story, is 0.30 mm at 0.25% drift, 2.50 mm at 0.50% drift, and 5.00 mm at 1.00 % drift.

Fig.9 shows the position of the yielding rebar at 0.50% and 1.00% drift. ○ indicates longitudinal reinforcement of wing walls, □ indicates longitudinal reinforcement of columns, ◎ indicates longitudinal reinforcement of beams, and ● indicates slab reinforcement. All of tensile longitudinal reinforcement of wing walls in 1st story has been yielded at 0.25% drift. The longitudinal reinforcement of beam members in 2nd, 3rd, and 4th story, the slab reinforcement in 2nd and 3rd story, and tensile reinforcement of columns has been yielded at 1.0% drift.



(a) 0.50 % total drift (b) ultimate state

Fig. 8 – Cracking pattern of the specimen



(a) 0.50 % total drift (b) 1.00 % total drift

Fig. 9 – Yielding position of the longitudinal rebar

Table 3 – Residual crack width (mm)

	Total Drift 0.25%		Total Drift 0.50%		Total Drift 1.00%	
	South Inner End	North Outer End	South Inner End	North Outer End	South Inner End	North Outer End
RF Beam	0.05	0.10	0.10	0.15	1.60	1.00
5F Beam	0.05	0.10	0.15	0.15	3.50	3.00
4F Beam	0.15	0.10	1.20	1.80	3.50	5.00
3F Beam	0.20	0.30	1.00	2.50	3.00	5.00
2F Beam	0.20	0.20	1.20	1.50	1.70	5.00
	1st story column	1st story panel	1st story column	1st story panel	1st story column	1st story panel
South Column	0.00	0.00	0.00	0.00	0.10	0.10
Center Column	0.15	0.20	0.25	0.60	1.20	1.50
North Column	0.10	0.00	0.30	0.05	0.40	0.20

4.3 Moment Curvature for Beams and Columns

Fig.10 shows the moment curvature of north and south beam in 2nd story. The average moment curvature is derived from the difference between axial deformations measured at top and bottom of the beams. The large curvature is obtained around the gap, and the plastic hinge deformation also concentrates on this location as designed. The curvature is relatively higher inside of the beam, so that the clear span of the member does not increase in large deformation. The effect of attached spandrel walls and mullion walls is not obvious in this figure. Fig.11 shows the moment curvature of 1st story columns. The inflection point of the columns is estimated by formula (1) in Seismic Evaluation of Existing Reinforced Concrete Buildings [5]. The curvature at 0 mm and 1000 mm height is the maximum value for the column in X1 frame and X2 frame (wall tensile direction). This height is consistent with pull out and lap splice (720mm height) of the longitudinal reinforcement in the wall end section. The large flexural crack is also observed at the same height in the test. The inflection points of those three columns are 2700 mm height, and does not change by the length of the attached wing walls. The mean value of the inflection point height for the columns with both side wing walls and one side wing wall approximates the test results roughly.

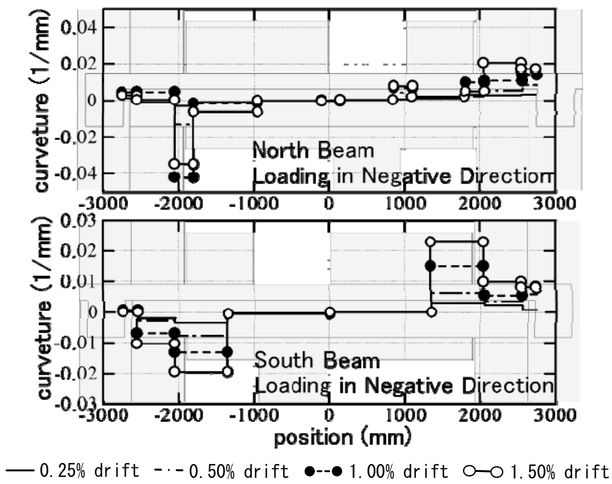


Fig. 10 – Moment curvature distribution of 2nd floor beams

$$h_{CW0} = h_{c0} + (h_{w0} - h_{c0}) \times (L_w / L) \quad (1)$$

Here, h_{CW0}; Height of the inflection point for columns with wing walls, h_{W0}; Height of the inflection point for multi-story shear wall, h_{C0}; Height of the inflection point for singular columns, L : span length, L_W : Width of wing walls

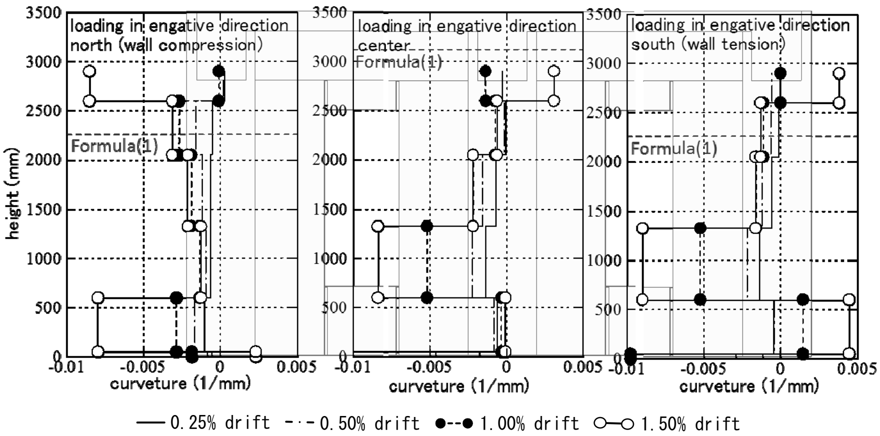


Fig. 11 – Moment curvature distribution of 1st story columns

4.4 Damage of windows

Photo.1 shows the damage state of the nonstructural elements using conventional fixing method at 2.0 % drift. Table 4 shows damage of those elements at unloading from 0.25%, 0.50%, and 1.00% drift. The windows are settled only in 1st story. The windows in the south frame of the 1st story is fixed with mortar as a conventional finishing joint, while the windows in the north frame of the 1st story is fixed with rock wool as a flexible finishing joint.

(a) Sliding type windows (b) Pressing-out type windows

Photo.1 – Damage of windows with conventional finishing joint (2.0% total drift)

Table 4 – Damage of the windows

Total drift	1st story drift	Flexible Joint (North)						Conventional Joint (South)					
		Sliding Type			Pressing-out Type			Sliding Type			Pressing-out Type		
		damage	open	lock	damage	open	lock	damage	open	lock	damage	open	lock
0.250%	0.187%	△	○	○	△	○	○	△	○	○	▲	○	○
0.500%	0.403%	▲	○	○	△	○	○	△	○	○	▲	▲	▲
1.000%	0.826%	▲	○	○	×	▲	×	×	○	×	×	×	×

○;operational, △; minor damage, ▲; damage (difficult), ×; severe damage (disable)

The damage on the window frames are observed at 0.25 % drift in both windows. As for conventional joint, it is disable to open at 0.50% drift in the pressing-out type window, and lock at 1.00% drift in the sliding type window. As for flexible joint, it is operational until 0.50% drift, but disable to lock at 1.00% drift in the pressing-out type window. The performance improves in the flexible method, but those windows are disable until 0.80% story drift (1.0% total drift) except for opening of the sliding windows with flexible joint.

5. Concluding remarks

The study shows the outline of the static loading test on the full scale five story reinforced concrete building at the Building Research Institute. The following conclusions may be drawn from the test results:

・The proposed damage control design realizes the high-strength and ductile moment frames utilizing wing wall attached to the same size column as a rigid zone of beams, and it verified the performance by the static loading test of the full-scale five story reinforced concrete specimen.

・The frames make a form of obvious beam side sway mechanism. The cracking on beams and slabs concentrated around gaps, and it restraint the damage on beam column joint. The crack on the floor slab is parallel to the transvers beam in a full width of the slab.

・Moment curvature of the beams concentrated around the gap. The inflection point of the columns with wing walls can be evaluated with a fomula of interpolation between moment resisting frame and wall frames.

・Windows installed in openings of 1st story are disable at the 1.00% total drift (0.80% story drift).

6. Acknowledgements

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