Seismic Behavior of Existing Wall-type Precast Reinforced Concrete Residential Buildings with New Openings

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SUMMARY:
In order to utilize a large number of existing wall-type precast reinforced concrete (WPC) residential buildings, a methodology to evaluate the seismic performance of such buildings is developed. Static pushover analysis models of existing 5-story residential buildings were created. The models consist of elastic line elements for wall panels and inelastic springs for relatively vulnerable joints. Full-scale experiments of the joints were conducted to investigate the pull-out tension properties, which impact on overall lateral building behavior. Using the analysis models, the ultimate seismic strength (base-shear) coefficient was determined at approximately 0.7 and the collapse mechanism is rocking of multi-story shear walls associated with shear failure of coupling beams and failure of the joints on the first floor. Creating new openings in shear walls may not change the collapse mechanism with the relatively vulnerable joint failure, and degradation of the lateral strength of the building may be limited.

Keywords: Shear walls, Precast reinforced concrete panels, Existing buildings, Pushover analyses, Connections

1. INTRODUCTION

Wall-type precast reinforced concrete (WPC) residential buildings, assembled using prefabricated concrete panels for the slabs and walls, were widely constructed during the 1960s and 70s in Japan to counter the serious housing shortage. The number of existing WPC residential units constructed pre-1980 is approximately 470,000. The prefabricated panels maintain good concrete quality, and their high seismic performance was confirmed in recent major earthquakes in Japan (Kobe 1995 and Tohoku 2011) with very limited structural damage (AIJ, 1998). Utilizing such structurally superior building stock is thus economically and environmentally preferable. However, they are not fully in use due to their small and uniform unit plans that do not suit modern living styles. Although the creation of new openings in the walls could expand the potential for plan changes during renovations, no design methodology for new openings has yet been developed.

In the authors' previous research, a half scale experiment for the shear wall panels was conducted, featuring the real design of a prototype 5-story building (Takagi *et al*., 2011A). The specimens were composed of a shear wall panel in the second story and part of the wall panels in the upper and lower stories, as well as part of the flange wall panels. Static pushover analysis models with inelastic springs for the joints were created for the shear wall and the properties of the springs were calibrated with the test results (Takagi *et al*., 2011B). In this research, models were developed for prototype 5-story residential buildings. Renovation schemes were studied and new openings were found to be needed in the walls between residential units. The collapse mechanisms and ultimate lateral strengths of the buildings were evaluated under a static seismic load in the short-side direction. Furthermore, the influence of new openings on the seismic performance of the building was studied.

2. COMPOSITION OF WPC RESIDENTIAL BUILDINGS

Figure 1 shows the typical floor plan of WPC residential buildings, which are typically 5-story or less, with no corridor connecting the residential units in each floor. Each stair is shared with every two units in each floor.

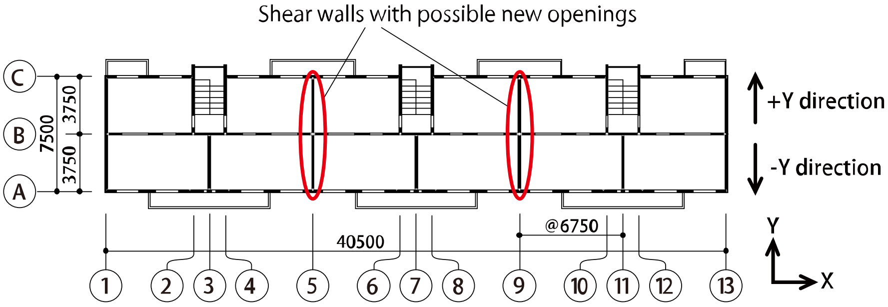

Figure 1. Floor plan of prototype WPC residential buildings

As shown in Figure 2, prefabricated wall and slab panels are assembled to construct WPC residential buildings. For the connections between the wall panels, steel plates and welded reinforcements are embedded in the upper and lower sides of the panels (Figure 3). The embedded steel plates are field-welded, and these connections are called “setting bases (SBs).” In addition, there are also connections with shear connectors on the vertical sides of the wall panels (vertical connections) as shown in Figure 4. The extended reinforcement is welded to the reinforcement of the horizontally adjacent wall panels, and the gaps between the panels are filled with concrete on site. In the gaps, reinforcement is vertically placed penetrating the slab levels. This reinforcement is known as a “vertically connecting reinforcement (VCR),” and functions to connect vertically adjacent walls. Figure 2 also shows coupling beams above the entrance door of each unit. They are not connections between prefabricated wall panels, but function as connecting elements under seismic lateral force.

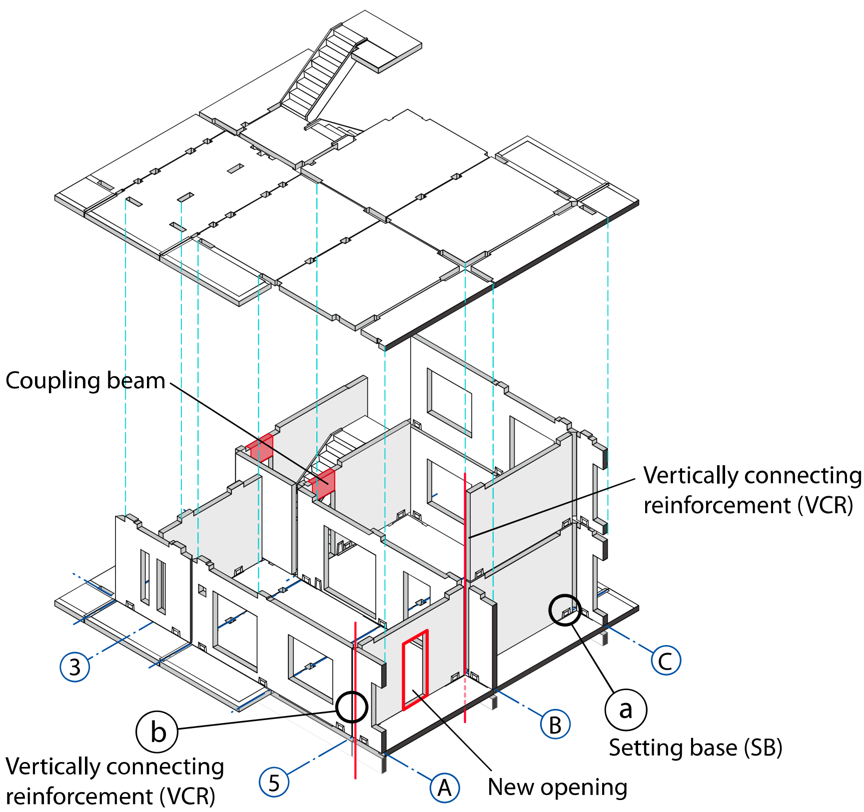

Figure 2. Composition of WPC residential buildings

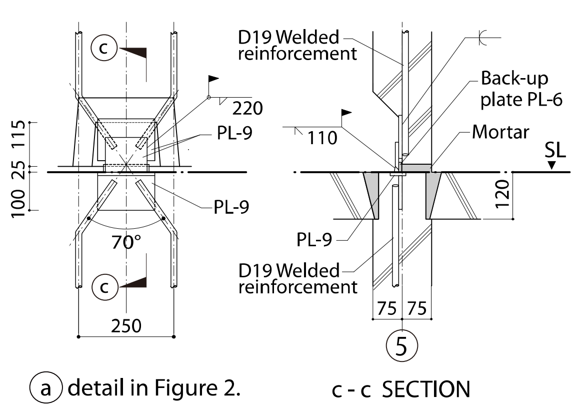

Figure 3. Setting base (SB)

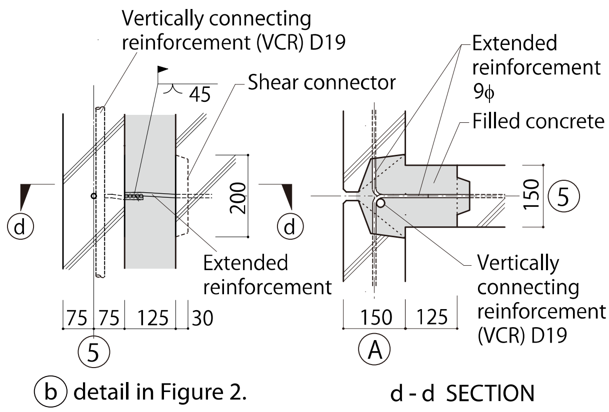

Figure 4. Vertical joint

3. NUMERICAL ANALYSIS MODELS

3.1. Model Structure

Two dimensional analysis models of 5-story existing prototype WPC residential buildings shown in Figure 5 are created. In order to investigate the collapse mechanism and ultimate lateral strength, including the influence of new openings in shear wall panels, analysis models are prepared for seismic loads in the short-side direction (Y direction in Figure 1), in which new openings between the residential units are needed. Identical frames are condensed into one frame in the analysis model and three independent frames are created as shown in Figure 5. The independent frame axes are 1, 2+3, and 5. Frame 2+3 is the combined frame of frames 2 and 3. Shear walls in these frames are connected with coupling beams above the entrance (Figure 2). Because the beams connect to Frame 3 via the flange wall in Frame B, pinned connections are adopted at the left end of the beams (Figure 5).

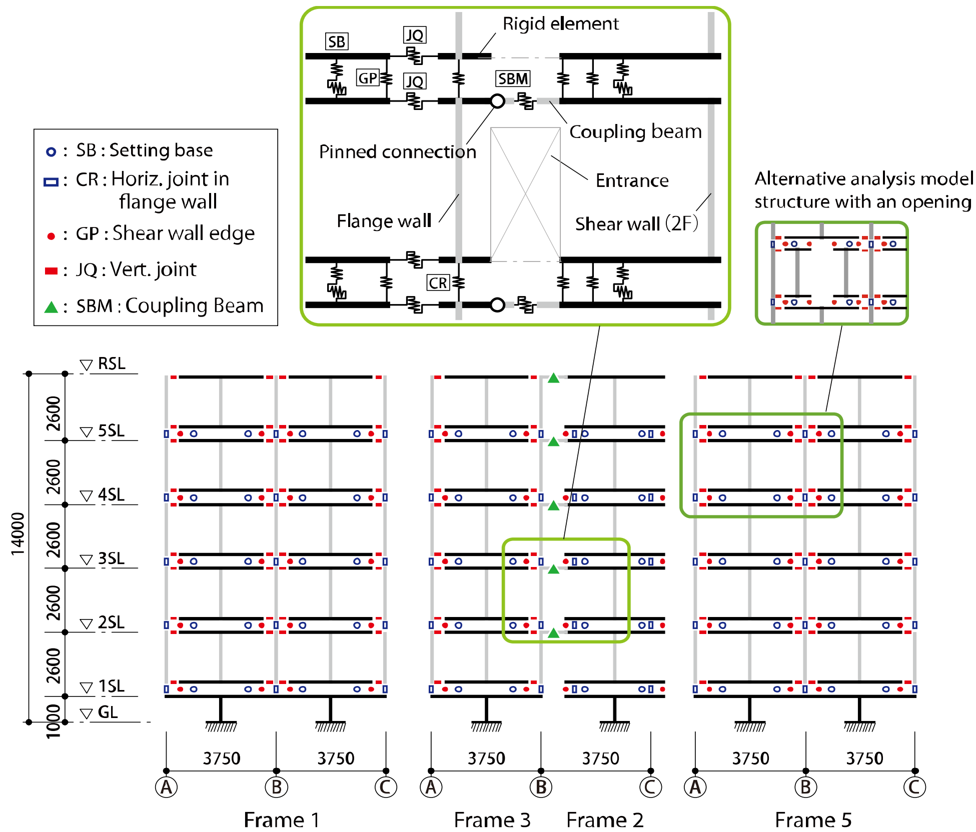

Figure 5. Analysis model structure

Independent frame groups are summarized in Table 1. Adopting the diaphragm condition, the lateral displacement at each floor is confined as the same. The foundation is assumed to be sufficiently strong and is excluded from the analysis model.

Table 1. Identical frame group

Identical frame axis	1, 13	2, 4, 6, 8, 10, 12	3, 7, 11	5, 9
Number of identical frames	2	6	3	2

3.2. Inelastic Springs

The analysis models consist of inelastic springs for the joints and elastic line elements for the wall panels. This modeling reflects the relative vulnerability of the joints, the evaluation of which is key for the overall structural behavior. Details of the property settings of the springs shown in Figure 5 are described in the authors' previous research (Takagi *et al.*, 2011A, 2011B, and 2012) and summarized in Table 2.

Table 2. Inelastic springs in analysis models

Spring	Direction*	Description
SB	X	Represents the sway deformation of shear walls at slab levels Perfect elastic-plastic bi-linear curve with high elastic stiffness and ultimate strength based on Mattock (1972)
	Y+	Represents the tension force and displacement relationships at SBs Tri-linear curve as shown in Figure 8
	Y-	Sufficient strength for compression at SBs Rigid elastic spring
CR	Y+	Represents the tension force and displacement relationships at slab levels at flange walls Tri-linear curve with twice the SB (Y+) curve added to the perfect elastic-plastic curve for VCR
	Y-	Rigid elastic spring
GP	Y-	Rigid elastic spring
JQ	X	Rigid elastic spring, representing condensed horizontal displacement at the shear wall and adjacent flange wall
	Y	Represents shear displacement at vertical connectors between the shear and flange walls Perfect elastic-plastic bi-linear curve with rigid elastic stiffness and ultimate strength based on Nakano (2001)
	R	Rigid elastic spring
SC	Y+	Represents the tension force and displacement relationships at slab levels at new reinforcing columns Perfect elastic-plastic curve with calibrated strength and rigid elastic stiffness
	Y-	Rigid elastic spring
SBM	X	Rigid elastic spring
	Y	Represents the shear deformation of coupling beams Quad-linear curve with rigid elastic stiffness, shear strength at cracking as 1/3 of the peak strength, which is defined as R=0.4% for the peak displacement, negative post-peak stiffness (0.5% of the elastic shear stiffness), and 40% of the peak strength for the residual strength
	R	Rigid elastic spring

* X: horizontal transition, Y: vertical transition (Y+: tension, Y-: compression), and R: rotation
Local coordinate axes of the springs are parallel to the global axes. Freedoms not shown in the table are not constrained.

Despite attempts to understand the behavior of the joints, there is a lack of information from the authors' previous half-scale shear wall experiment and past research to obtain a thoroughly rigorous definition of the various inelastic spring properties, taking into account a number of influential parameters. Therefore, this research focuses on providing reasonably rational analysis models for prototype WPC buildings and understanding characteristics of their seismic behavior.

3.3. Shear Property of Coupling Beams

Preliminary simulations indicated that the collapse mechanism was rocking of the multi-story shear walls, accompanied by failure of the first floor SB joints. It was also found that the coupling beams above the unit entrance (Figure 2) had significantly enhanced ultimate lateral strength (approximately 20 percent). Referring to the original structural drawings of the building, shear failure is the dominating failure mode of the beams. To evaluate the influence of the coupling beams, inelastic shear springs are placed at their midpoint (Figure 5). The springs have rigid elastic stiffness in global horizontal (X) and rotational (R) directions, and inelastic stiffness in a vertical (Y) direction. The inelastic vertical stiffness is defined as shown in Figure 6. The force-displacement relationships are a quad-linear curve with rigid elastic stiffness (elastic shear deformation simulated by line elements of the coupling beams), shear strength at cracking as 1/3 of the peak strength (Q_{su}), which is defined as R=0.4% for the peak displacement, negative post-peak stiffness (0.5% of the elastic shear stiffness, which is shown as K_0 in Figure 6), and 40% of the peak strength for the residual strength. The peak strength (Q_{su}) is calculated as 129kN (average shear stress of 1.7N/mm^2) using Arakawa (minimum) equation (Arakawa, 1970).

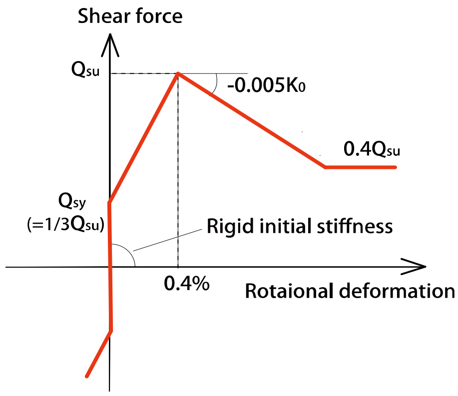

Figure 6. Shear spring (SBM) property of coupling beams

3.4. Tension Property of SB

Because the collapse mechanism is accompanied by rocking of the multi-story shear walls with tensile failure of SBs on the first floor, the tensile property of the SB connections significantly influences the ultimate lateral strength of the building. In the authors' previous experimental study, which investigated the tensile properties of SBs, welding rupture was observed, although yielding of the connecting reinforcement was the designed failure mode (Takagi *et al.*, 2011A). 9mm thick steel plates embedded in SB (Figure 3) were 4.5 mm thick in the half-scale specimen and the difficulty of welding to the thin plates was considered a possible reason for the unexpected result.

To enhance the accuracy of the inelastic tension spring property of the SBs, full-scale experiments were conducted on the SB joints. A specimen consists of an SB and upper and lower story wall panels nearby. Two specimens with connecting reinforcements of varying size were tested. Figure 7 shows the specimen of the SB tension test, which contains embedded steel plates and connecting reinforcement and precast concrete walls around the SB. Concrete sections of the upper and lower two pieces were separately cast and assembled with welding. The sizes of the connecting reinforcements in the prototype buildings were D22, D19, and D16, with a larger size used in the lower story. Two types of test with the connecting reinforcement of D19 and D16 (Tests 1 and 2, respectively) were conducted. (D22 was not available with SD295 steel.) The material properties of the specimen are summarized in Table 3. Monotonic tension force was exerted on the specimens and the force-displacement relationships as shown in Figure 8 were obtained. In Test 1, concrete spalling around SB was observed at a vertical displacement of 15mm and one of the two connecting reinforcements failed at 37mm. The maximum tension force was 265kN, which was 8% greater than the nominal tension strength of the reinforcement. Similar damage was observed in Test 2 (D16).

Referring to the SB experiments, tension properties of the SB are defined as shown in Figure 8. Considering the combined effect of tension and in-plane or out-of-plane sliding forces, which are simultaneously subjected to the connections, the maximum strength of the spring was reduced to approximately 75% of the tested strength. A reduction rate equivalent to the SB strength in the shear wall test (Takagi *et al.*, 2011) was used.

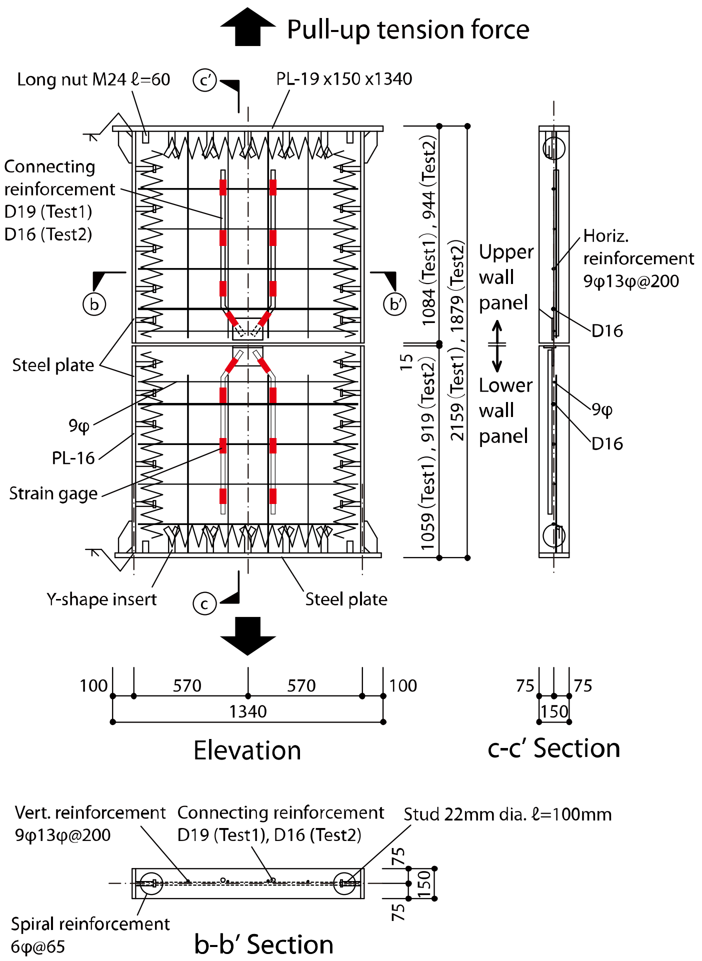

Figure 7. SB tension test specimen

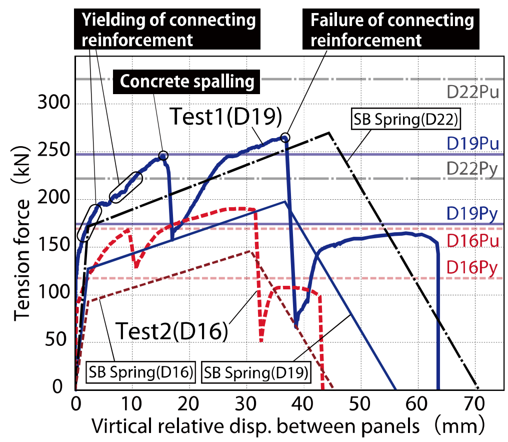

Figure 8. SB tension force-displacement relationships

Table 3. Material properties of SB test specimens

CONCRETE

	F_c (N/mm²)	σ_B (N/mm²)	σ_T (N/mm²)	εco (x10⁻³)
Test 1	27	63.6	4.77	3.18
Test 2		64.9	4.23	3.30

STEEL

	Spec.	Location	σ_y (N/mm²)	σ_u (N/mm²)	Yielding strain (x10⁻⁶)
PL-9	SM490	SB Steel plates	392	541	1919
9φ	SR235	wall reinforcement	※363	※477	1771
13φ	SR235	wall reinforcement	※331	※451	1615
D16	SD295	SB connecting reinforcement wall horiz. reinforcement	348	497	1881
D19	SD295	SB connecting reinforcement	366	527	1912
Welding rod	D5016	welding at SB plates	※500	※570	—

※: mill-sheet or catalog values

Fc : concrete strength in design, σ_B : concrete compression strength, σ_T : concrete tension strength, εco : strain at compression strength, σ_y : steel yield stress, and σ_u : steel tension strength

4. ANALYSIS RESULTS

Conducting the static pushover analysis, the relationships between the lateral force and displacement as shown in Figure 9 (+Y dir.) were obtained. The vertical and horizontal axes indicate the ultimate seismic strength (base-shear) coefficient (C_{Q1}) and rotational deformation, respectively. The rotational deformation (R) is defined as the lateral displacement at roof level with respect to the height from the first floor to the roof. The maximum C_{Q1} is determined at 0.67 at R=0.2% (+Y dir.) and 0.72 at R=0.3% (-Y dir.). Figure 10 shows the collapse mechanism at R=1.0%. The mechanism involves rocking of the multi-story shear wall associated with the shear failure of coupling beams and joints on the first floor. The solid and hollow round marks in the figure indicate inelastic deformation of the joint springs beyond the elastic limit and peak strength, respectively. In the +Y direction, yielding (elastic limit) and failure (peak strength) of SB joints on the first floor are observed at around R=0.15% and 0.7%. Also, shear failure of the coupling beams is seen at around R=0.25%. C_{Q1} peaks at 0.67 with R=0.2%, however, virtually all the strength remains up to R=1.0%, meaning no ductile behavior is observed. In the –Y direction, building damage is similarly observed.

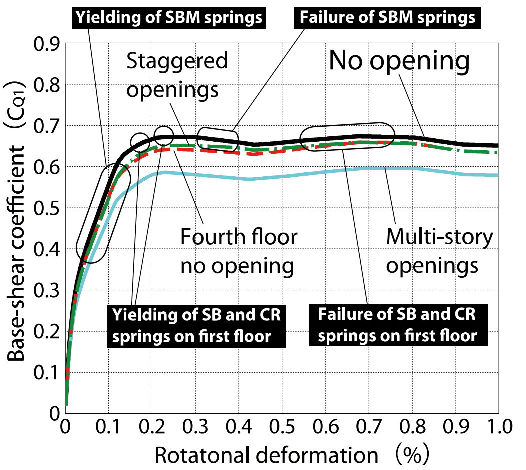

Figure 9. Lateral force displacement relationships

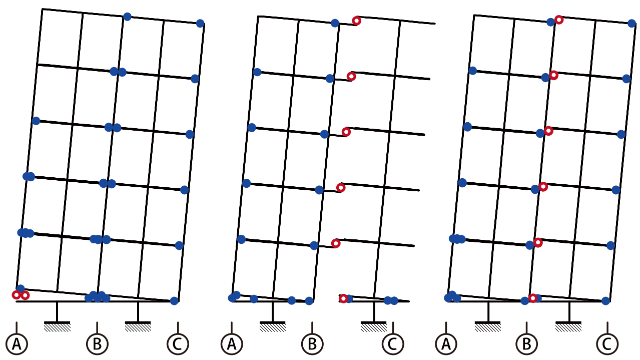

Figure 10. Collapse mechanism

5. INFLUENCE OF NEW OPENINGS

Creating a new opening splits an existing precast wall panel into two, and the remaining wall above the opening is not strong enough to behave as a coupling beam. With the opening, the analysis models were modified as shown in Figure 5. Without reinforcement of the opening, the contact (GP) spring, which is stiff in compression and free in tension, is placed at the top and bottom of the side of the opening. Pushover analyses with various opening patterns in Frame 5, which is identical to Frame 9, were conducted (Figure 11). The opening in the first floor (Figure 11(a)) results in a limited decline in ultimate lateral strength (less than 3 percent), because the rocking collapse mechanism of multi-story shear walls remains unchanged. The maximum shear stress in the wall is 1.9 N/mm^2, where the shear stress is calculated as the shear force in the line elements of the walls divided by the cross-sectional area and excluding that of the connecting flange walls. The maximum stress is considered to be lower than the shear stress at shear failure of the walls (Takagi *et al.*, 2011B). This result implies that the relative vulnerability of the joints dominates the lateral strength and collapse mechanism. Consequently, this opening is not influential to the building behavior. On the other hand, multi-story openings in identical locations on each floor (Figure 11(b)) result in separation of the multi-story shear wall and significantly reduce the lateral strength of the building (12 percent). However, the remaining upper single-story shear wall without opening (Figure 11(c)) prevents the separation of the shear wall and reduces the decline in strength (one percent). In addition, multi-story openings in a staggered pattern (Figure 11(d)) influence the building behavior differently from those in identical locations in the plan. The strength deterioration in this case is two percent.

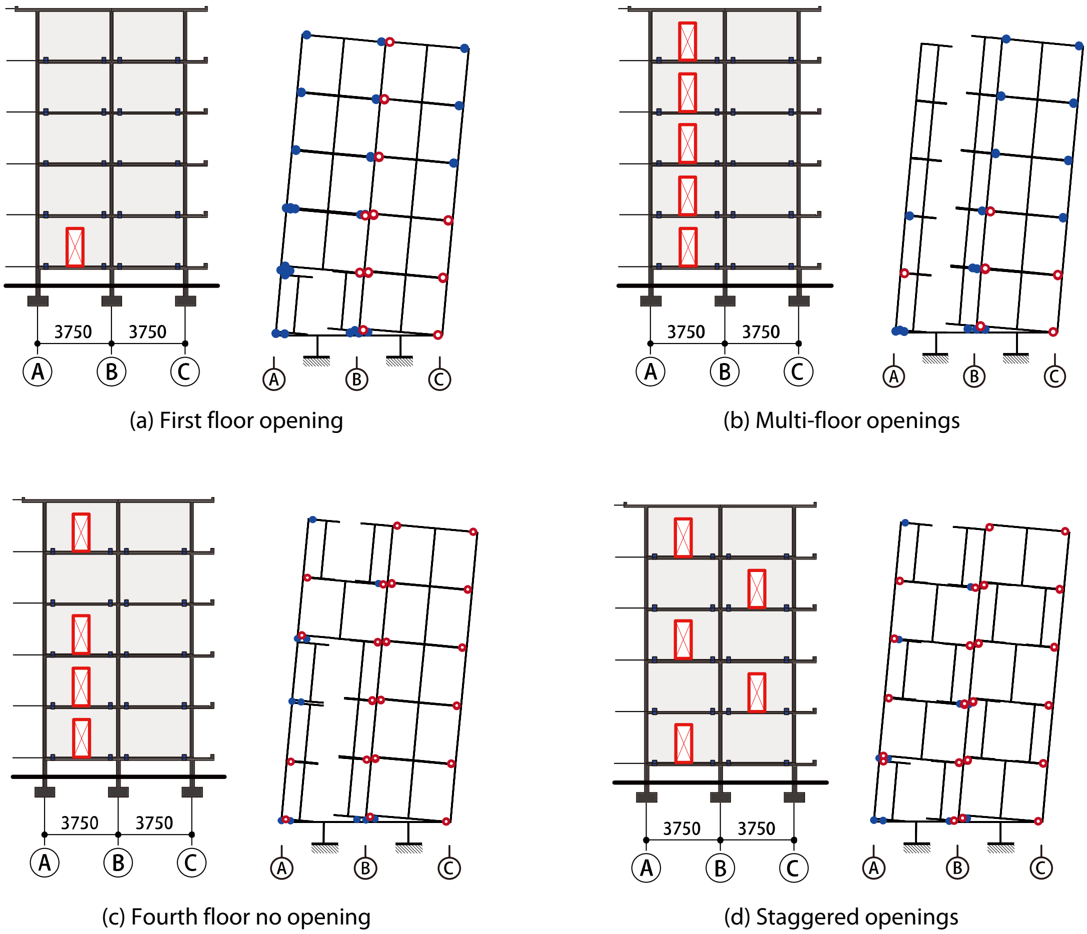

Figure 11. New openings in Frame 5 (and 9) and deformed shape (R=1.0%)

6. CONCLUSION

A large number of wall-type precast reinforced concrete (WPC) residential buildings exist in Japan maintaining high structural quality. In order to utilize this type of building stock, creating new openings in the precast shear walls expands the potential for plan changes during renovations and the structural performance evaluation of the buildings is needed. The objective of this research is to provide a method for the evaluation and the knowledge obtained is summarized as follows:

(1) Numerical analysis models of prototype WPC residential buildings were developed to evaluate the collapse mechanism and ultimate lateral strength. Models were prepared for the short-side direction, in which new openings are needed in the shear walls in the buildings. Shear wall panels and connections between the panels are modeled as elastic line elements and inelastic springs, respectively. The tension properties of the horizontal joints called setting-base (SB) influence the building performance, and were defined based on full-scale tension test conducted.

(2) Using the analysis models developed, static pushover analyses were conducted. The ultimate seismic strength (base-shear) coefficient was determined at approximately 0.7 and the collapse mechanism was rocking of the shear walls accompanied by failure of the coupling beams and the connections on the first floor.

(3) Analysis models with new openings in shear walls were also developed. It was found that the influence on ultimate lateral strength may be limited, because the connections were relatively vulnerable compared to the precast wall panels and the failure mechanism remained

unchanged. Creating new openings at an identical location on every story would vertically split the multi-story shear wall and reduce the ultimate lateral strength by 12%. However, remaining single wall without any new opening significantly reduces the decline in strength, because the wall without an opening functions as a coupling beam.

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