Performance Evaluation of Shear Walls in Existing Wall-type Precast Reinforced Concrete Residential Buildings with New Openings

J. Takagi[1], S. Minami[2], and K. Kitayama[3]

[1] Division of Architecture and Urban Studies, Tokyo Metropolitan University,
1-1 Minami-Osawa Hachioji City Tokyo, Japan;
PH +81-42-677-2798; FAX +81-42-677-2793; email: jtakagi@tmu.ac.jp
[2] Tokyo Metropolitan University (same affiliation and address as above);
PH +81-42-677-1111 (ext.4723); FAX +81-42-677-2793; email: minamis@tmu.ac.jp
[3] Tokyo Metropolitan University (same affiliation and address as above);
PH +81-42-677-2802; FAX +81-42-677-2793; email: kitak@tmu.ac.jp

ABSTRACT

Wall-type precast reinforced concrete (WPC) residential buildings, which are assembled using prefabricated concrete panels for the slabs and walls, were widely constructed during the 1960-70s in Japan. The number of existing WPC residential units constructed before 1980 is approximately 470,000. Although these buildings are of high quality in terms of structural condition, they are not fully utilized due to their small and uniform unit plans that do not suit modern living styles. Creating new openings in the walls could widen the possibility of plan changes during renovation; however, a design methodology for new openings, including the addition of structural reinforcement, has not been developed for this unique structural system. In this research, reinforcement design for new openings was developed and improvement of seismic performance was examined in half-scale cyclic loading tests. Numerical analysis models were also developed and the behavior of the walls was well simulated.

INTRODUCTION

Wall-type precast reinforced concrete (WPC) residential buildings (Figure 1) were widely constructed in Japan from the middle of the 1960s to counter the serious shortage of housing. Figure 2 shows the annual and accumulative number of WPC units constructed. Approximately 470,000 units were built before 1980. Most of them still exist and maintain good physical condition with precast (PCa) concrete slab and wall panels. Also, their high seismic performance was confirmed from the limited damage observed after the Hanshin-Awaji Earthquake in 1995 (AIJ, 1998). The unit plans for this type of residential building were highly standardized, and each unit area is rather small for modern family use. The buildings typically have five stories without elevators. Stairs were constructed between every two units in the plan and there are no corridors connecting floors, providing both privacy and natural ventilation during summer. With its inconveniences, this housing stock has not been fully utilized despite its good structural condition.

Figure 1. WPC residential buildings

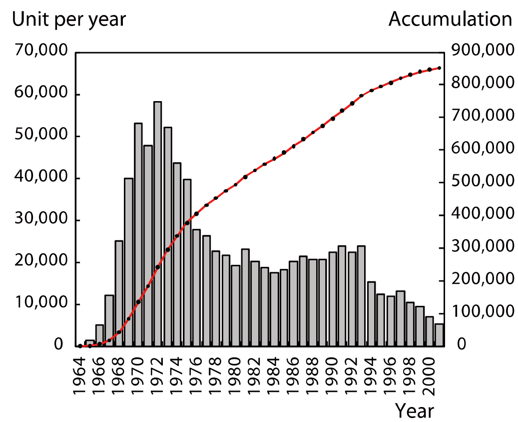

Figure 2. Number of WPC units

In order to activate the housing stock, a renovation design study team in which the authors participated was organized. Barrier-free schemes are proposed considering various aspects including financial feasibility and legal restrictions (Figure 3). In order to provide various unit plans suiting modern life styles, new openings in existing walls are needed in the schemes. However, a methodology for reinforcing the openings in this unique structural system has not yet been developed.

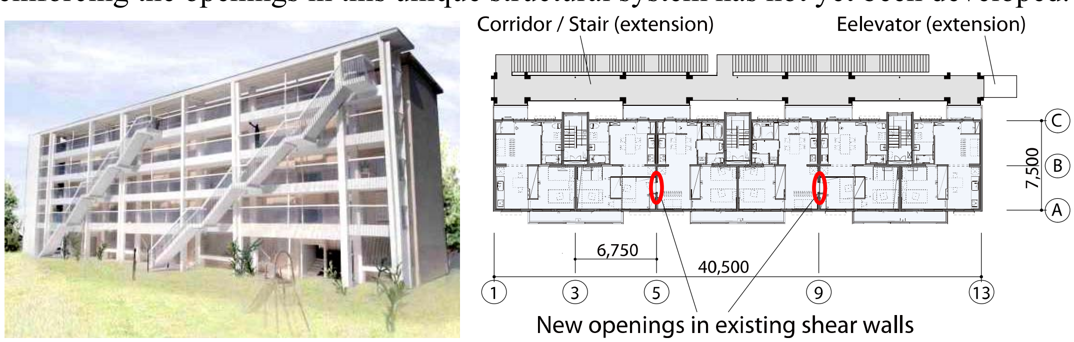

Figure 3. Renovation scheme

In this research, reinforcement design schemes for the new openings in the existing wall panels are developed, and their seismic performance is experimentally evaluated. Also, numerical analysis models for the wall tests are created using inelastic springs for the connections between wall panels.

WPC RESIDENTIAL BUILDINGS

As shown in Figure 4, prefabricated wall and slab panels are assembled for the construction of 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 5). The embedded steel plates are field-welded, and these connections are called "setting bases (SBs)." In addition, there are connections with shear connectors on the vertical sides of the wall panels (vertical connections) as shown in Figure 6. 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 called "vertically connecting reinforcement (VCR)," and this works to connect vertically adjacent walls.

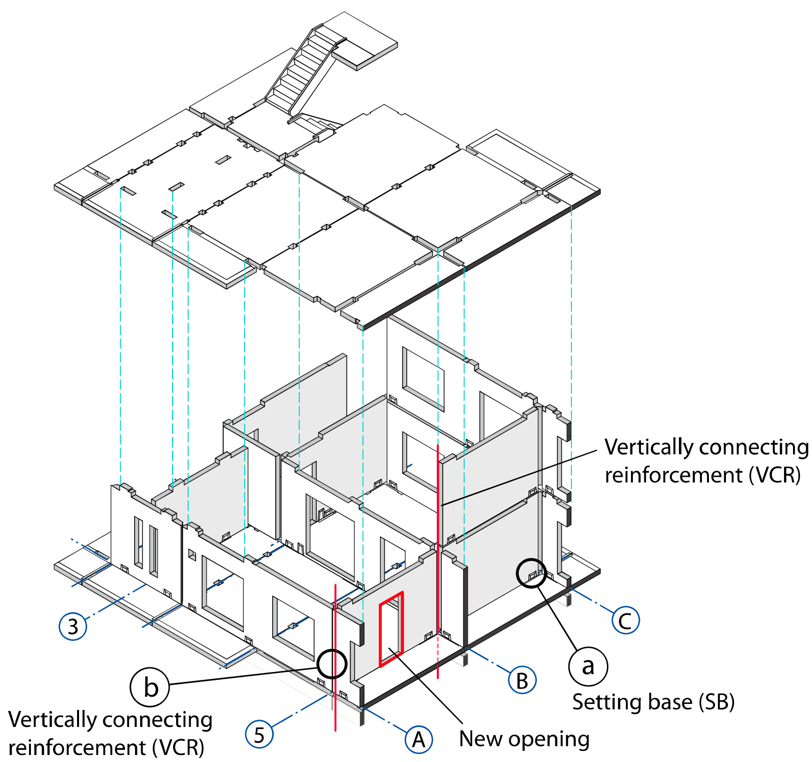

Figure 4. WPC structure

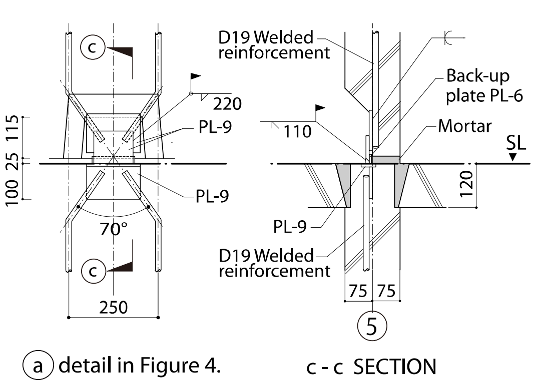

Figure 5. Setting base (SB)

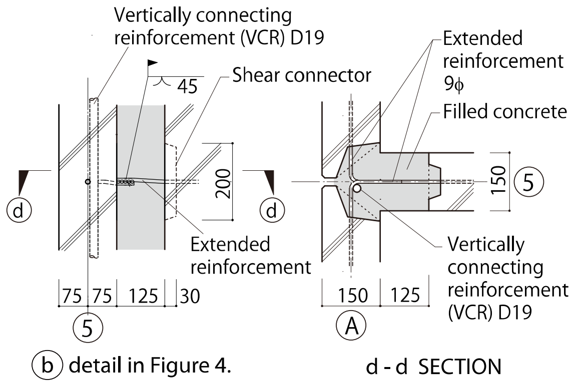

Figure 6. Vertical joint

REINFORCEMENT FOR NEW OPENINGS

Experiencing a significant number of major earthquakes, Japanese seismic design codes have been developed with revisions to design specifications. Consequently, not every existing building necessarily meets all of the current specifications. These existing buildings are not illegal; however, structural reinforcement to meet requirements is needed in the case of major renovation. Generally, this reinforcement is technically difficult and significantly costly. On the other hand, if the renovation is relatively minor, where the number of (partially) structural members demolished is limited, the current design specifications do not apply under the condition that each member (or part of the building) maintains equal or greater structural performance due to the reinforcement involved in the renovation.

Because the seismic performance of the existing WPC residential buildings is generally high, creating new openings in some of the wall panels may not be essential. Depending on the target seismic performance criteria, no reinforcement may be needed. Studying the seismic performance of WPC buildings with new openings is an interesting subject (and this is our future work), and with much consideration given to rapid practical application of the research outcome, our first research objective was to develop reinforcement design schemes for the walls as required by most jurisdictions. (A performance-based design approach, including evaluation of the whole building, takes more time as several issues such as research development, design review, and authorization must be tackled.)

In WPC shear wall panels, there are more than two SBs on the upper and lower sides. The SBs carry pull-up force induced by the overturning moment (OTM) under the seismic lateral load. In the renovation schemes, new openings are placed at the center of walls, not disturbing the SBs, as shown in Figure 7. The new opening subdivides the wall panel into two. If rocking of these wall panels is the major deformation mode as shown in Figure 7, the resistance capacity of the wall can be significantly reduced. Also, a part of the wall above the new opening is not strong enough to be a coupling beam. Using reinforced concrete (RC) or steel (S) members, reinforcement design schemes are developed by strengthening connections to the walls in the lower story and coupling beams above the new openings.

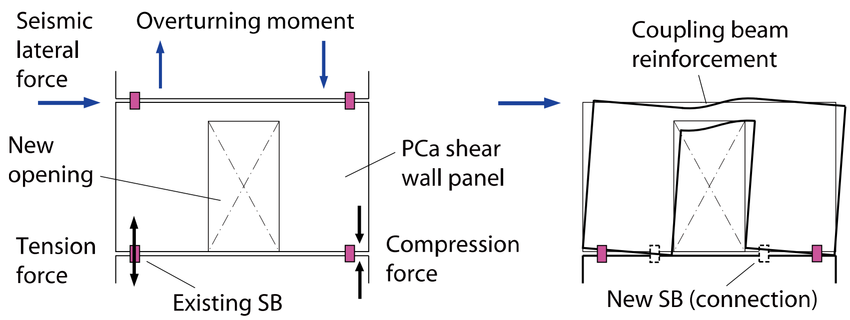

Figure 7. Seismic load resisting system in WPC shear walls

TESTING PROCEDURE

A half-scale experiment was conducted for the shear wall panel on the second floor in the five-story building shown in Figure 3, where there are PCa wall panels on the upper and lower stories and where the seismic lateral load is the greatest. The specimens are composed of a shear wall, part of the walls of the first and third stories, flange walls, and slabs. Stubs are also created at the top and bottom of the specimens for loading. The test parameters are openings in the shear wall, shear-span ratio (H/W), and RC or S reinforcement. Specimens with a small H/W have openings in the first and third stories, while those with a large H/W do not. Eight specimens were created: one with no opening, two with no reinforcement, two with RC reinforcement, and three with S reinforcement. Figure 8 shows the C5S specimen (RC reinforcement with a large H/W) and N5M (no reinforcement with a small H/W). The new RC columns are placed on the side of the new opening in C5S. The columns connect to the new beams under the existing PCa slab panels. A list of specimens with fundamental information is shown in Table 1.

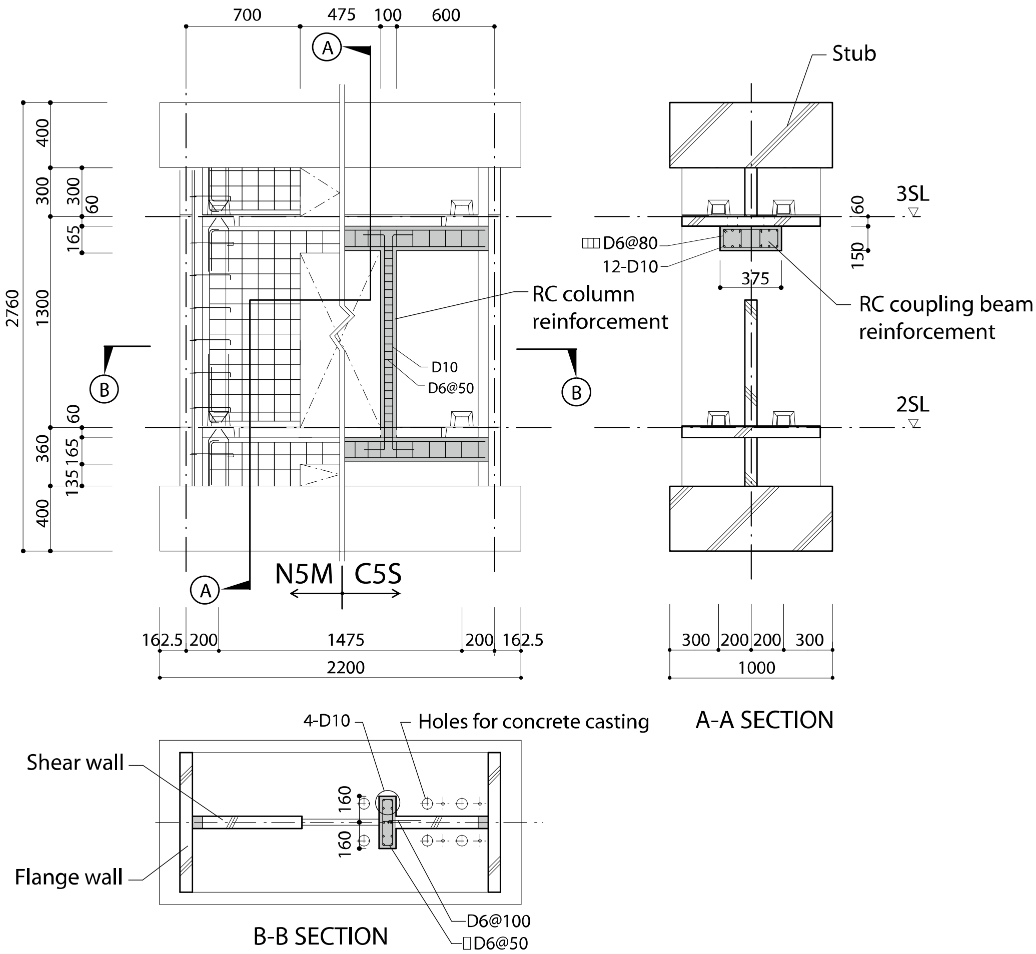

Figure 8. Shear wall test specimens (N5M and C5S)

Table 1. Specimens and fundamental information

Specimen	H/W	Reinforcement	Concrete strength *1) (N/mm^2)			Maximum strength *2) (kN)		Initial stiffness *3) (kN/mm)	Failure mode *4)	Description
			${}_P\sigma_B$	${}_J\sigma_B$	${}_C\sigma_B$	+dir.	-dir.			
W5	1.85 (large)	none	58.6	44.8	-	101	103	166	F	Damage around 2SL SBs
N5S			67.0	48.0	-	105	110	53	F	Failure of 3SL SBs
C5S		RC	58.0	55.2	74.4	154	139	190	F	Damage to first-floor walls under reinforcing columns
S5S		S	66.3	45.7	-	124	117	97	F	
B5S			50.2	59.5	-	113	106	64	F	Damage around 2SL SBs
N5M	1.17 (small)	none	60.8	57.8	-	136	132	40	F	
C5M		RC	52.8	78.8	74.3	289	271	189	F/S	Shear failure (significant damage) of shear walls
S5M		S	51.1	52.4	-	220	235	101	S	

*1) Concrete strength (N/mm^2) on the test day (${}_P\sigma_B$: PCa panels, ${}_J\sigma_B$: joint concrete, and ${}_C\sigma_B$: reinforcing members); *2) Maximum lateral strength (kN) (+dir: positive direction, and -dir: negative direction); *3) Initial stiffness defined as secant stiffness with the origin and strength at R=+0.025%; *4) Failure mode (F: SB failure, and S: shear failure of wall)

Figure 9 shows the loading system of the specimens. Under a constant vertical load, which represents the dead and live load (shown as D in Figure 9), static incremental cyclic load Q (=Q1+Q2) is applied. Coupled vertical force V, which is equivalent to the OTM, is also loaded in proportion to lateral force Q. In order to investigate the influence of the OTM on the behavior of the wall, two shear span ratios (shown as H/W in Figure 9) were prepared: 1.85 (large) and 1.17 (small).

Controlled cyclic lateral displacement is applied to the specimens for rotational deformation, R =0.025%, 0.05%, 0.1%, 0.2%, 0.33%, 0.5%, 0.67%, 1.0%, and 2.0%, with two cycles each except R =0.025% with one cycle, where R is defined as the lateral displacement of the center of the upper stub divided by the height from the top of the bottom stub.

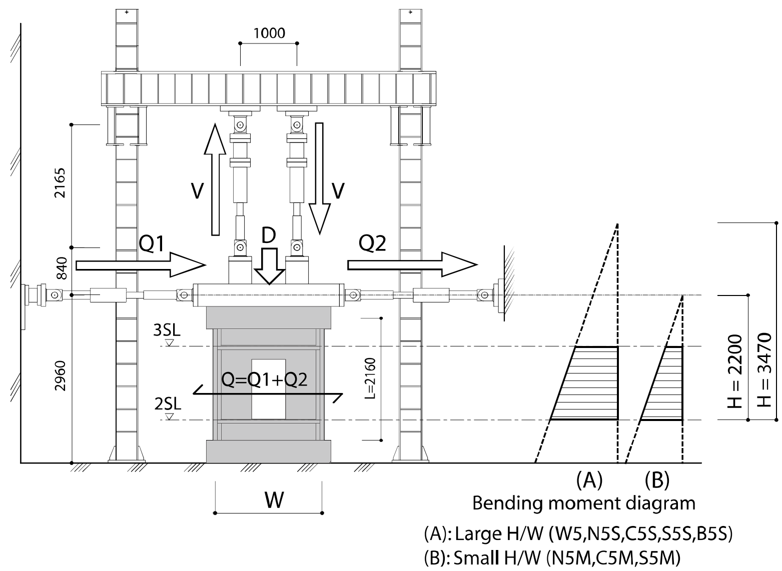

Figure 9. Loading system

TEST RESULTS

Shown in Figure 10 are the crack patterns in the shear walls after loading in W5, N5S, C5S, and C5M, out of the eight specimens. In W5 and N5S, minor cracks were observed in the wall panels and damage is concentrated around the horizontal connections (the SBs and VCRs). Rocking of the wall with failure of the horizontal connections is the overall failure mode in these tests. In N5S, shear cracks were observed; however, they did not directly trigger strength degradation. Regarding behavior of the flange walls, damage was limited to the panels and was observed only at the connections on the upper and lower sides of the panels in all cases.

In C5S, where the new opening is reinforced with RC members and the

H/W is large, several shear cracks were observed in the shear wall panels. Major horizontal cracks were also generated in the wall panel on the first floor. The pull-out force transferred to the wall from the new RC column via the new connected beam was large enough to cause cracks in the wall under the new column. Thus, the connection detail between the wall and the new beam needs to be improved. Although shear cracks were observed in C5S, the primary failure mode is also rocking of the shear wall with failure of the horizontal connections. In C5M, where the new RC columns on the side of the opening penetrate the floor slabs and are embedded in the upper and lower stubs, the walls failed in shear. Because the new columns vertically tighten the wall panels and the H/W is relatively small, failure of the horizontal connections was less significant compared with other tests with a large H/W.

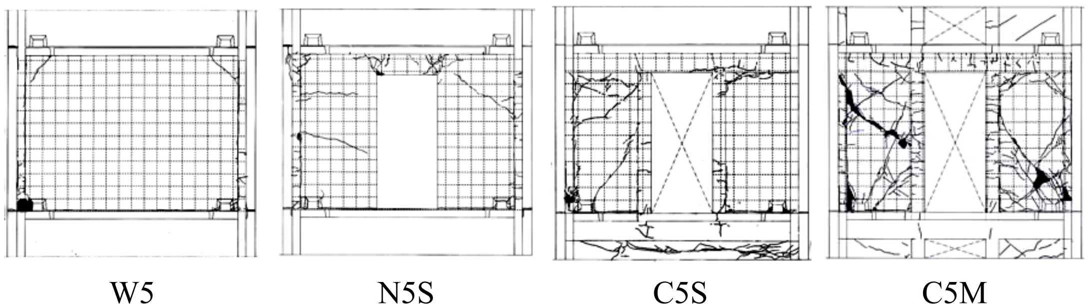

Figure 10. Crack patterns in shear walls

Figure 11 compares the relationships between the lateral force and rotational deformation in the three tests on W5, N5S, and C5S. The difference in the maximum lateral strength of W5 (no opening) and N5S (no reinforcement for the opening) is limited. This is because the failure mode in these tests is rocking with failure of the SBs under a large H/W. Shear failure of the wall was not critical and, consequently, the opening in the wall did not reduce the maximum lateral strength. Contrarily, the initial lateral stiffness and equivalent viscous damping ratio of N5S is approximately 30% and 50% those of W5, respectively; therefore, the difference in their energy dissipation capability is significant.

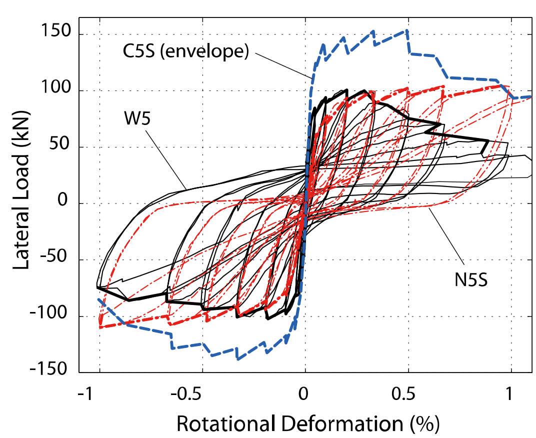

Figure 11. Test results

In C5S, the maximum lateral strength was 140-150% that of W5 and N5S. Similarly, enhancement of seismic lateral strength was observed in other test cases with reinforcement. Although further improvement is needed such as for the connection of the reinforcing and the existing members, the reinforcement design was able to provide equal or greater seismic performance compared to WPC walls without openings.

NUMERICAL SIMULATIONS

In order to simulate the behavior of the WPC wall in the test, numerical analysis models were created. Figure 12 shows two-dimensional models of W5, C5S, and C5M. The models are composed of elastic line elements for the wall panels and inelastic springs for the connections. Shear springs in the shear walls were also included in the models for the small H/W test, where shear failure was observed. The elastic modulus of the concrete was calculated based on the tested concrete strength according to AIJ (2010). In some wall tests, it is observed that rocking of the shear walls is the primary failure mode and behavior of SBs significantly influences the overall behavior of the wall.

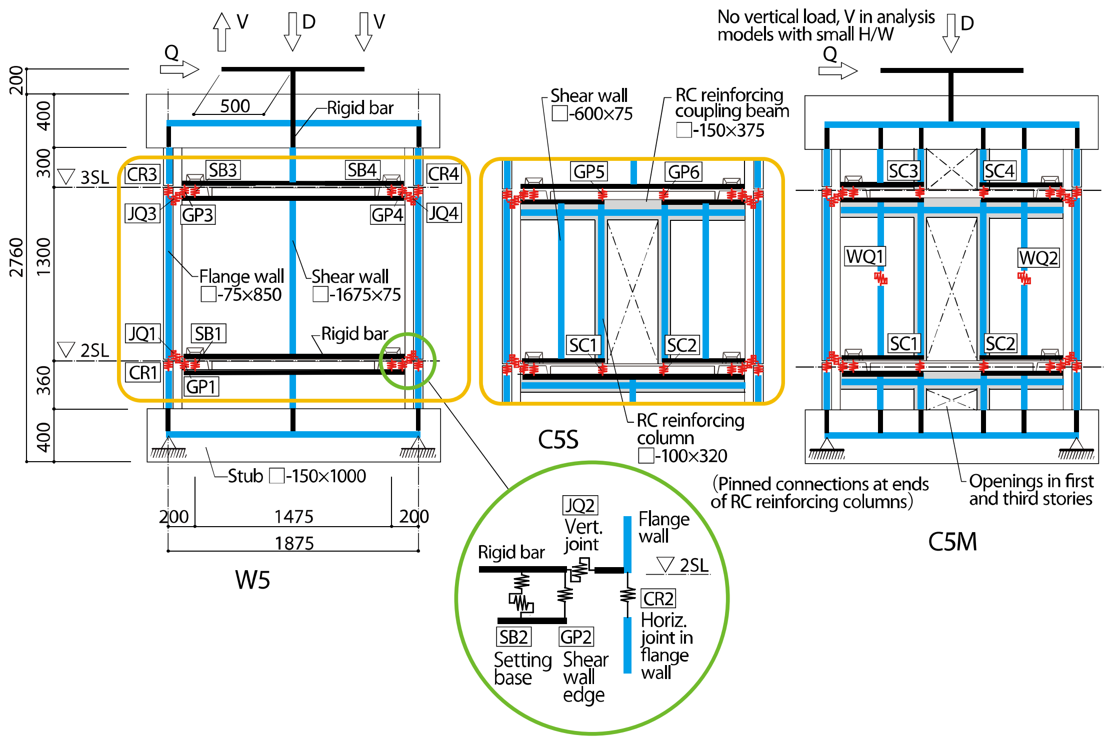

Figure 12. Analysis models

In order to investigate the SB properties, especially pullout tension force and vertical displacement relationships, additional tests were conducted. Shown in Figure 13 is the half-scale specimen of the SB test. Vertical displacement of the specimen composed of an SB and its vicinity in the WPC wall panels on two stories was enforced. The test was performed under two types of loading: one-way tension and cyclic tension-compression. In order to separate the wall panels into two stories, a steel plate was inserted at their boundary. It was expected that the plate would minimize the vertically connecting tensile strength between the concrete panels using a potential manufacturing process.

Shown in figure 14 are the relationships between the controlled vertical displacement and load on the specimens. Peak strength was observed at approximately 8 mm from the vertical displacement with yielding of the reinforcement and fracture of the plate welding. As shown in Figure 14, a tri-linear tensile property of SBs is defined for the analysis models. Calibrating the shear wall simulation with the test, the peak strength is reduced to 75% of the more accurately approximated tri-linear model. A possible physical reason for this reduction is the combination of lateral and vertical force in the wall test.

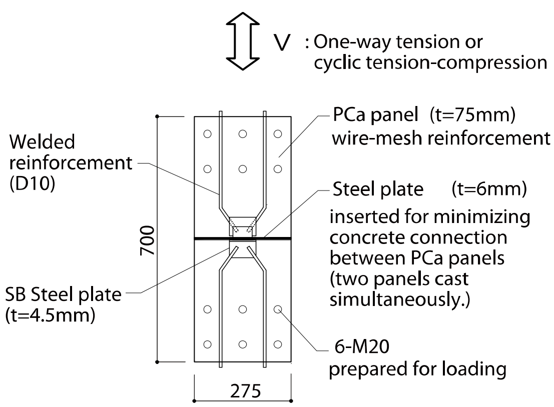

Figure 13. Specimen of SB tests

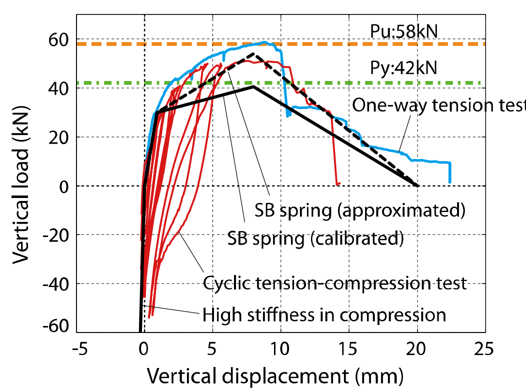

Figure 14. SB test and spring

Other properties of the inelastic springs in the analysis models of the walls are summarized in Table 2. The SB, CR, GP, and JQ spring properties are consistent in the analysis models, while the SC and WQ properties are evaluated for each test with opening reinforcement. Using these simulation models, displacement control static pushover analyses were conducted (MIDAS). Figure 15 shows the analytical results superimposed on the test data. The analyses reasonably simulate the envelope of the cyclic loading test. It is noteworthy that the analyses simulate the failure mode of the WPC walls with reasonable order and accuracy, as well as the strength.

Table 2. Inelastic springs in analysis models

Spring	Direction*	Description
SB	X	Represents 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 14
	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 wall and flange wall Perfect elastic-plastic bi-linear curve with rigid elastic stiffness and ultimate strength based on Nakano (2001)
SC	Y+	Represents 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
WQ	X	Represents shear deformation of shear walls Quad-linear curve with rigid elastic stiffness (elastic shear deformation simulated by line elements of the walls), shear strength at cracking as 1/3 of the peak strength, which is defined through calibration with the test, R=0.4% for the peak displacement, negative post-peak stiffness (0.5% of the elastic shear stiffness), and 1/2 of the peak strength for the residual strength
	Y	Rigid elastic spring
	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.

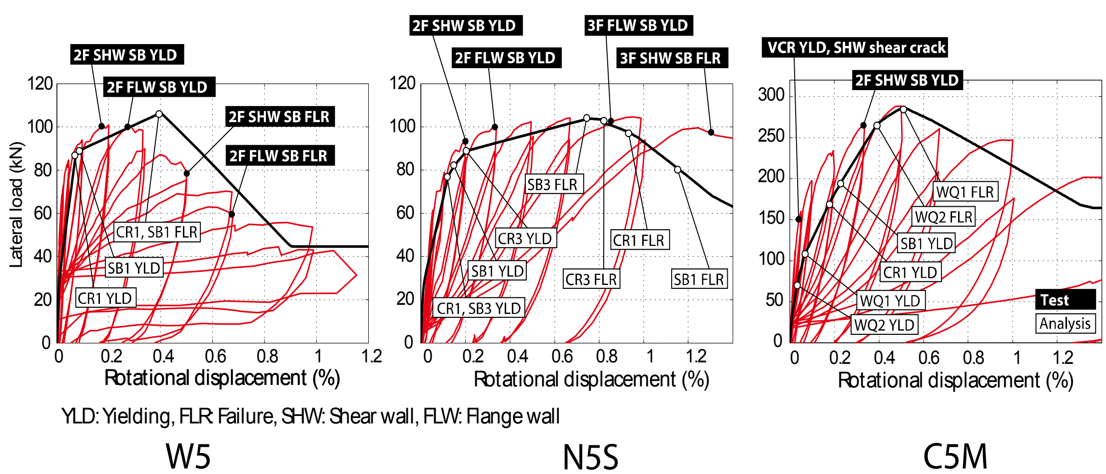

Figure 15. Analysis and test results

CONCLUSIONS AND FUTURE WORK

In this research, reinforcement design for new openings in existing shear walls in wall-type precast reinforced concrete (WPC) residential buildings was developed and improvement of seismic performance was examined through half-scale cyclic loading tests. Furthermore, two-dimensional numerical analysis models, which are composed of line elements for the wall panels and inelastic springs for the connections and shear behavior of the walls, were created. Using the models, the behavior of the tested walls was well simulated. As future work, analysis models are to be further developed for the evaluation of whole building seismic performance.

REFERENCES

Architectural Institute of Japan (AIJ). (1998). *Report on the Hanshin-Awaji Earthquake Disaster Building Series* vol. 2 (in Japanese).

Architectural Institute of Japan (AIJ). (2010). *Standard for Structural Calculation of Reinforced Concrete Structures* (in Japanese).

Mattock, A. H. and N. M. Hawkins. (1972). “Shear Transfer in Reinforced Concrete□Recent Research,” *PCI Journal*: 3-4

MIDAS Information Technology Co., LTD., Midas GEN ver. 761.

Nakano, K. and Y. Matsuzaki. (2001). “Additional Method of Shear Resistances in Precast Concrete Connections,” *J. Struct. Constr. Eng AIJ* no. 550: 151-158 (in Japanese).

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