SEISMIC FORCE RESISTING MECHANISM OF THE MULTI-STORY PRECAST CONCRETE SHEAR WALL SUPPORTED ON PILES

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ABSTRACT :

When cantilever structural walls are subjected to severe earthquakes, rocking and sway motions are likely to take place at the base of the walls. Then the foundation structure, which consists of first floor slabs, foundation beams, and piles, supporting the rigid structural wall may uplift under certain circumstances and the resulting stress state may differ from the analytical prediction based on simple design assumptions. This is because the interaction between the superstructure and the foundations has not been thoroughly studied for its complexity. This study aims to experimentally clarify the lateral load resisting mechanisms considering the interaction between a structural wall, foundation beams, slabs and piles. In order to investigate the lateral load resisting mechanism of the structural wall – foundation assemblage, two types of specimens were tested. One of the specimens had monolithic wall and the other specimen had precast wall which had vertical and horizontal joints around precast wall panels.
Cracks across the wall panels and along the member interfaces were a major source of deformation of the structural wall and two specimens showed different deformation modes. Nevertheless, hysteresis loops of two specimens were similar with respect to stiffness, capacity and energy dissipation. Based on the experimental results, the load resisting mechanism of the foundation beam was clarified and numerical models were proposed for different stages of deformation and damage of the wall – foundation assemblage.

KEYWORDS: structural wall, foundation beam, pile, precast concrete wall, interaction

1. INTRODUCTION

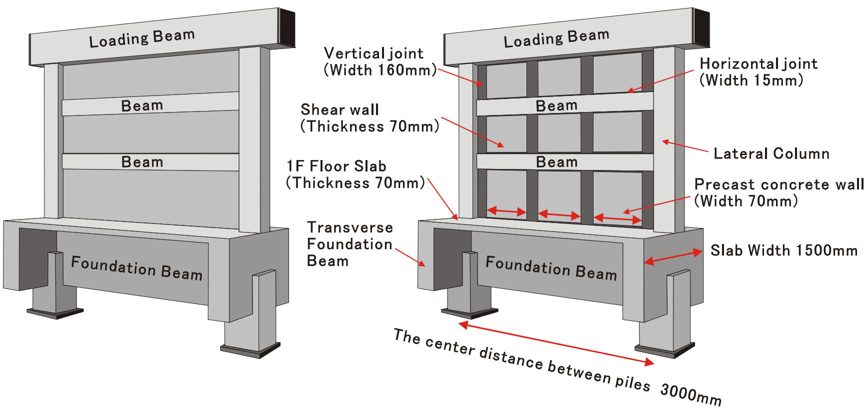
Typical Japanese mid-rise and high-rise residential buildings have multiple bay RC moment resisting frames in the longitudinal direction and single bay structural wall systems in the transverse direction. When cantilever structural walls are subjected to severe earthquakes, rocking and sway motions are likely to take place at the base of the walls. Then the foundation structure, which consists of first floor slabs, foundation beams, and piles, supporting the rigid structural wall may uplift under certain circumstances and the resulting stress state may differ from the analytical prediction based on simple design assumptions. This is because the interaction between the superstructure and the foundations has not been thoroughly studied for its complexity [1,2].
Foundation beams under structural walls transmit earthquake induced forces from the structural walls to the soil through the piles. In the cases that the foundation beams under structural walls have enough stiffness and strength to resist earthquake-induced forces, the interaction between the structural walls and the foundation beams doesn't matter and it is possible to simulate seismic behaviors of each member separately in the transverse direction. On the contrary, without sufficient stiffness and strength of the foundation beams, it is necessary to consider the behavior of the structural wall and pile foundation system monolithically. Because the interactions have not been clarified yet, there is no procedure to check whether the foundation beams have

enough strength and stiffness during the seismic response.
This study aims to experimentally clarify the lateral load resisting mechanisms considering the interaction between a structural wall, foundation beams, slabs and piles. It enables to establish more rational design procedures for each structural component. In order to investigate the effect of the difference of the lateral load resisting mechanism on the seismic behavior of the pile foundation, two types of specimens were tested. Difference of these specimens was the construction methods of the structural walls. One of the specimens had monolithic wall and the other specimen had precast wall in which vertical and horizontal joints were arranged.

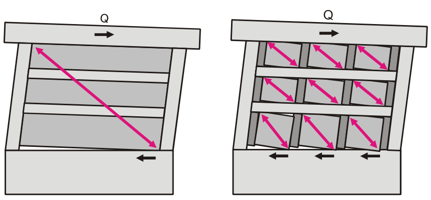
2. EXPERIMENTAL SETUP

2.1. Specimen

Figure 1 shows the configuration of specimens. They consisted of the bottom three story of a structural wall with a foundation beam, first floor slab, and piles. They were scaled to 25%. The configuration of specimens was determined from typical fourteen story residential buildings in Japan.
Difference of these specimens was construction method of the structural walls. The structural wall of MNWL was cast monolithically and that of PCWL had vertical and horizontal joints to simulate the behavior of the precast wall system. Basically the same performances as the monolithic walls are necessary for the precast concrete members. For example, shear keys are provided at joints. In this study, slippage at the vertical and horizontal joints was permitted in order to form different resisting mechanism from that of monolithic structural walls. Figure 2 shows the image of the lateral load resisting mechanism of each specimen. The difference can have influence on the behavior of the pile foundations.
Vertical joints extended the height of each story and divided them three. Horizontal joints were arranged along the top and bottom edge of the PCa wall panels. They were filled with joint mortal. Permitting the slippage at the joints, these joint surfaces of the PCa wall panels intended to be cast smoothly. Other differences of these specimens were the arrangement of the vertical reinforcement of the wall and concrete strength. But the amount of the vertical reinforcement was same in both specimens.



(a) MNWL **(b) PCWL**
Figure 1. Specimen configuration



(a) MNWL **(b) PCWL**
Figure 2. Difference of the lateral load resisting mechanism

For both specimens, the structural walls were designed to fail in flexure. Flexural behaviors of the structural walls have much influence on the variation of the lateral load resisting mechanism according to the deformation of the structural walls. The point of contraflexure for the piles was fixed at 1250 mm from their top, even though the depth of the contraflexure point in practice varies with soil conditions, and the intensity of the axial and lateral forces acting on the piles. The first floor slabs extended 750 mm on either side of the centerline of the walls. The structural walls and the slabs had a thickness of 70mm. The square piles were designed to remain elastic throughout the test, so that the lateral load could be increased until the structural wall failed. The piles extended to midheight of the foundation beams and were without caps for simplicity even though piles in practice are circular and have solid pile caps. Material properties are shown in Table 1 and the types of reinforcement are listed in Table 2.

Table 1. Material properties

(a) Concrete

		Compressive strength (MPa)	Tensile strength (MPa)	Young's modulus (GPa)
PCWL	Foundation beam, Pile	40.6	3.23	27.7
PCWL	Column, Beam	50.0	3.09	27.3
PCWL	Wall	43.9	2.55	26.9
PCWL	Joint mortar	64.5	3.55	28.3
MNWL	Foundation beam, Pile	45.7	3.41	25.9
MNWL	Wall, Column, Beam	60.3	3.32	30.4

(b) Reinforcement

	Yield strength (MPa)	Tensile strength (MPa)	Young's modulus (GPa)
D6	377	532	179
D10(SD295A)	378	511	183
D10(KSS785)	919	1078	201
D13	351	505	175
D16	337	502	191
D22	341	525	183

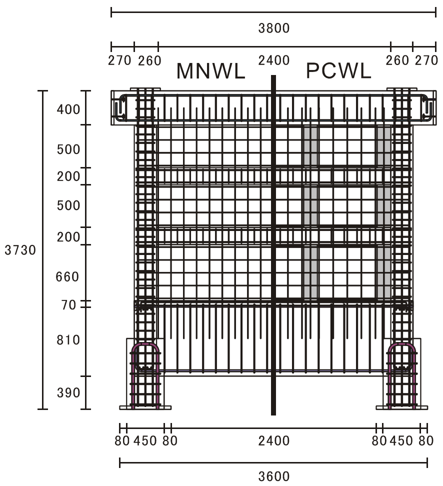


Figure 3. Reinforcement arrangement (unit: mm)

Table 2. Types of reinforcement

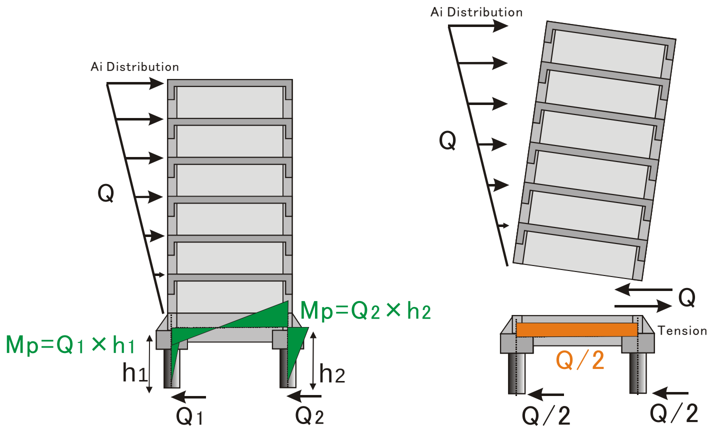
Member	Type of bars		Steel ratio(%)
Column (260×260mm)	Longitudinal	8-D13	1.50
	Transeverse	2-D10@100 (KSS785)	0.549
Beam (140×200mm)	Longitudinal	4-D10 (SD295A)	1.18
	Transeverse	2-D6@150	0.302
Shear Wall (70mm)	Vertical	D6@150	0.302
	Horizontal	D6@150	0.302
Pile (440×440mm)	Longitudinal	8-D32	3.28
	Transeverse	2-D13@120	0.480
Foundation Beam (150×880mm)	Longitudinal	4-D22	1.26
	Transeverse	2-D10@150 (SD295A)	0.634
Slab (70mm)	Longitudinal	D6@150	0.302
Transverse Foundation Beam (260×880mm)	Longitudinal	8-D16	0.738
	Transeverse	2-D10@150 (SD295A)	0.366
Loading Beam (350×400mm)	Longitudinal	8-D16	1.36
	Transeverse	2-D10@150 (SD295A)	0.272

In Japanese design guideline [3], the required amount of the longitudinal reinforcement in the foundation beam under structural wall is determined separately based on the following two external forces as shown in Figure 4.

- Moment from piles (Mp)
- Axial force Q/2 (tensile force) [Q is the total lateral force applied to the structural walls.]

As shown in Figure 4 (b), large rotation of the shear wall makes the shear force Q transfer through the limited area under compression. In general, the lateral force applied to piles that are subjected to tensile axial force is smaller than the lateral force applied to piles that are subjected to compressive axial force. Axial force Q/2 is given as the possible maximum axial force in the foundation beams at the ultimate state. As for Mp in Figure 4 (a), foundation beam is modeled as a line element and Mp is calculated by the lateral load applied to the piles and the distance between the contraflexural point of the piles and the midheight of the foundation beam.
This design procedure is based on the engineering judgment, but the forces acting on the foundation beam from structural wall isn't considered. As shown in Figure 5, from the structural wall, vertical tensile force of the shear reinforcement, vertical compression force of the concrete strut and moment from the compression column base are transferred in practice. In order to simulate the stress condition of the foundation beam in detail, it is necessary to consider these forces that vary depending on the deformation of the structural wall.
In this study, the amount of longitudinal reinforcement in the foundation beams was set smaller than the requirement of the Japanese design guideline [3]. Figure 6 shows moment distributions and reaction forces at Q=362kN. At this time structural walls yield in flexure and moment applied to the foundation beams from piles was 297kN·m. Flexural yield strength of the foundation beams ignoring contributions of peripheral members

was 230kN·m. The ratio of the foundation beam strength to the structural wall strength was 230/297=0.78. According to this design guideline, the yielding of the foundation beams precedes the yielding of the structural walls.



(a) Moment from piles (Mp) **(b) Axial force (Q/2)**

Figure 4. Design of foundation beam

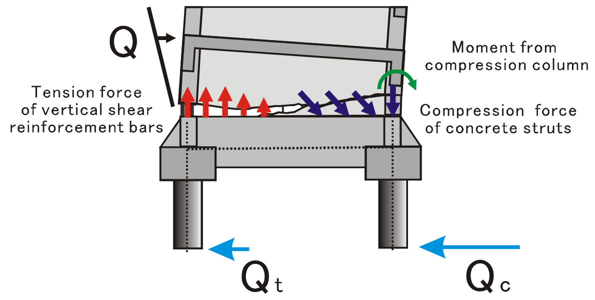


Figure 5. Forces acting on foundation beam from structural wall

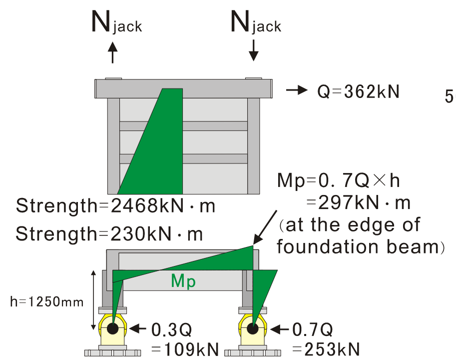


Figure 6. Moment distributions and reaction forces at Q=362kN

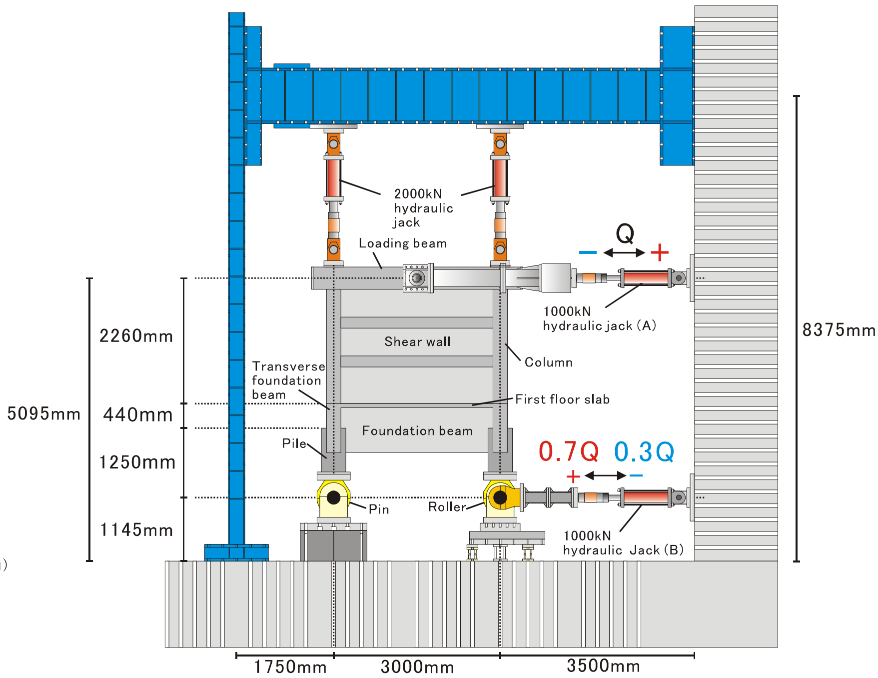


Figure 7. Loading system

2.2. Loading System

As shown in Figure 7, lateral load, Q, was applied statically to the loading beam on the top of the wall using a 1000kN horizontal hydraulic jack (A). Two 2000kN vertical hydraulic jacks were adjusted to create appropriate column axial forces at shear wall base, N1 and N2, which are a linear function of lateral load Q to simulate loading conditions of the prototype fourteen-story shear wall system during earthquakes.

$$N_1 \ \mathit{and} \ N_2 = \pm 2.27Q + 353 \text{ (unit:kN)} \qquad (1)$$

At the roller support, 0.7Q was applied horizontally to the pile when the pile was under compression and 0.3Q was applied to the piles when the pile was under tension the hydraulic jack (B) in the opposite direction to the hydraulic jack (A). As shown in Table 3, the load was applied two cycles at each preselected load level until flexural shear cracks developed in the structural walls to some extend. Then the displacement control was used with two cycles at each preselected displacement in Table 3. The first story drift angle is hereinafter called R. Finally MNWL and PCWL were loaded monolithically to +4.1% and -3.6% respectively.
In this experiment, it was difficult to directly measure the drift angles of the structural walls because of the rotation and the deformation of the pile foundation. So they were calculated from the flexural deformation and the shear deformation of the structural wall. They were measured with the multiple displacement gauges placed at the structural wall.

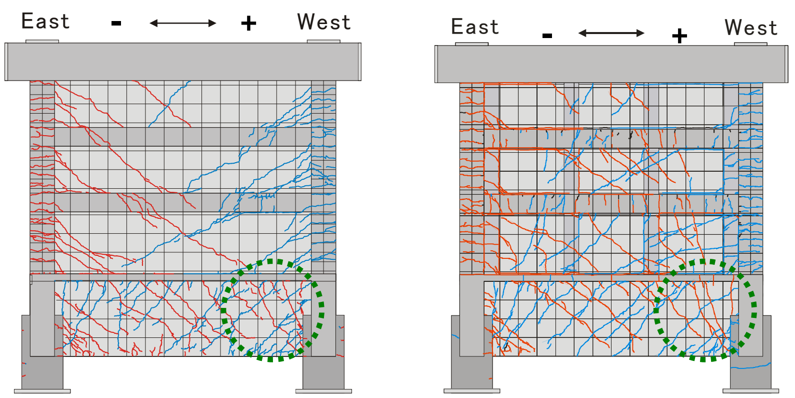
Table 3. Loading Protocol

	Lateral Load, ±Q (kN)			First Story Drift Angle, ±R (%)						
PCWL	150		250	0.09	0.16	0.33	0.51	0.68	0.82	1.5
MNWL	150	200	250	0.08	0.18	0.36	0.51	0.68	0.84	1.6

3. TEST RESULTS

3.1. Observed Damage and Deformation Mechanism

Figure 8 shows the crack distributions. Flexural cracks took place at tensile columns and propagated to flexural shear cracks in the wall panels. PCWL had cracks along the vertical and transverse construction joints and in the beams. The number of cracks in PCWL wall panels is greater than that of MNWL wall panels. Hence it was expected that the stiffness of PCWL is smaller than that of MNWL.
The shear cracks penetrated the slabs transversely and extended to the foundation beams in both specimens. As the deformation of the structural wall propagated, these cracks in the foundation beam near the compression pile became dominant and opened up as schematically shown in Figure 9. At this stage, the wall and the right pile together with the right top corner of the foundation beam made a solid assemblage rotated almost rigidly about Point P. When the upper longitudinal reinforcement of the foundation beam yielded, the rigid rotation of the assemblage propagated without the increase of lateral load. This mechanism is different from the one assumed in Figures 4 and 6 and plays an important role to simulate the stress condition of the foundation beam. It is discussed in detail in Section 3.3. Although the crack patterns of wall panels differed in two specimens, the rotation of the assemblage was equally observed in both specimens. Crack patterns of the foundation beams were similar to each other in two specimens.
Finally MNWL and PCWL were loaded monolithically to +4.1% and -3.6% respectively, but compression failure of the cover concrete at the compression column bases was hardly observed in both specimens.



(a) MNWL (b) PCWL

Figure 8. Crack distribution (R=0.8%)

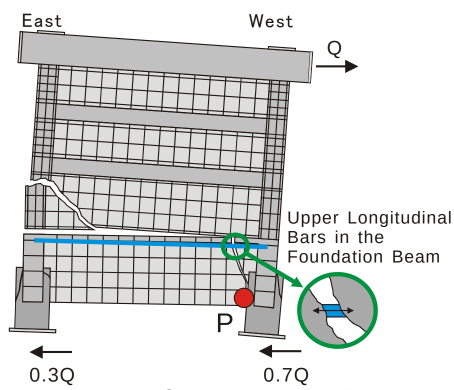
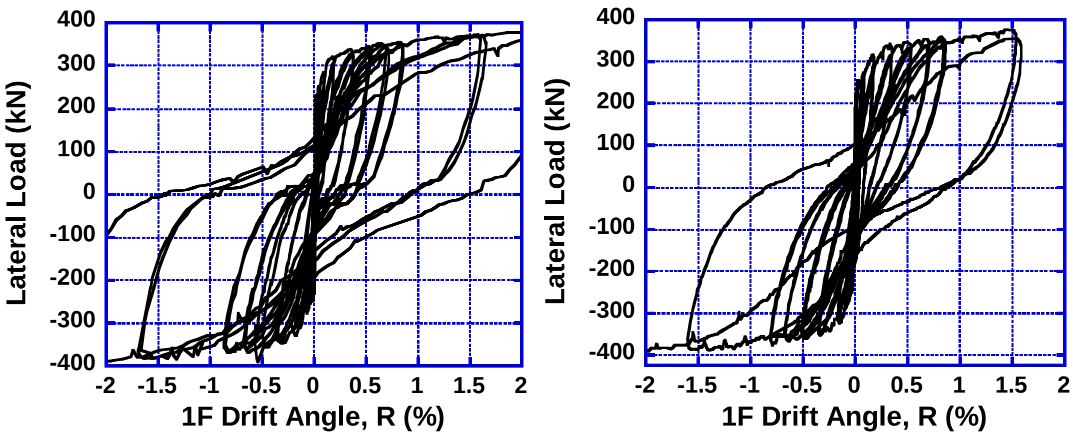


Figure 9. Deformation mechanism of the specimens

3.2. Lateral Load-Drift Angle Relations

Figure 10 shows lateral load – first story drift angle relations. Hysteresis loops of two specimens are similar with respect to stiffness, capacity and energy dissipation. In order to study the effect of different crack patterns of wall panels, the drift angle was decomposed to two contributions, flexural contribution and shear contribution. The decomposed at three different loading stages are listed in Table 4. It can be seen that flexural contribution in PCWL is nearly the same as that MNWL for small drift angle but the flexural contribution decreases at loading stage (b) and (c). This is because the relative slips along the construction joints greatly contributed the increase of shear deformation of the first story. However, the effect of cracks and slips along the construction joint did not appear in the load drift relation in Figure 10. These specimens were designed to fail in flexure, so the shear stress level at construction joints was not large enough to drive the rotation of the each PCa wall panel in PCWL, as shown in Figure 2(b). It was also because that the boundary beams and columns were large enough to give sufficient constraint on the precast wall panels of the specimens. If the size of the boundary beams and columns were smaller, the confining effect would decrease resulting in the softer lateral load – drift angle relations. In this experiment, the difference of the construction methods didn't have large influence on the seismic behavior of the structural wall and pile system.



(a) MNWL (b) PCWL

Figure 10. Lateral load – first story drift angle relations

Table 4. Flexural deformation and shear deformation at representative loading stages

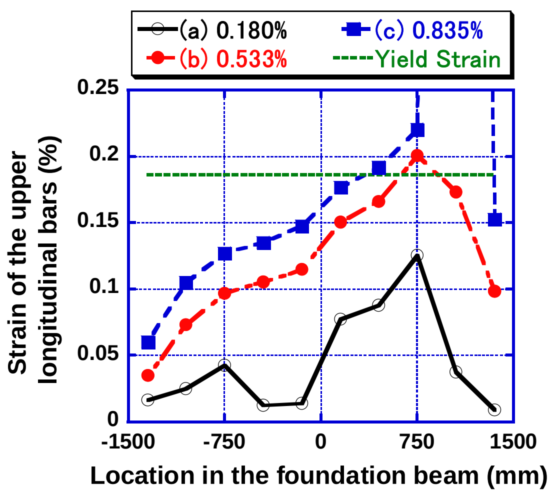
	Loading Stages	Flexural Deformation (%)	Shear Deformation (%)	Drift Angle (%)	Lateral Load (kN)
MNWL	(a)	0.046	0.134	0.180	319
	(b)	0.136	0.397	0.533	341
	(c)	0.203	0.632	0.835	353
PCWL	(a)	0.034	0.135	0.170	314
	(b)	0.077	0.436	0.513	344
	(c)	0.135	0.715	0.849	351

3.3. Strain Distributions of Longitudinal Reinforcement in Foundation Beams

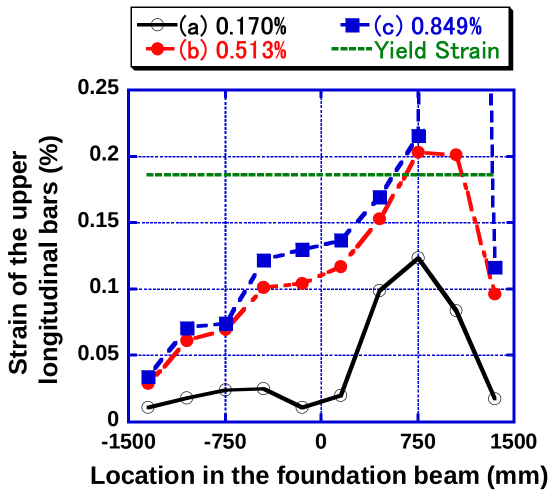
Figure 11 (I), (II) shows the strain distribution of longitudinal reinforcement in the foundation beam at three representative loading stages in Table 4. Location in the foundation beam in Figure 11 (I), (II) is illustrated with Figure 11 (III). From experimental results in Figure 11 (I), (II), large strain was measured in the region where cracks opened largely as schematically shown in Figure 8.

The tensile strain was measured in the whole span of the foundation beam. The strain distributions of the upper longitudinal reinforcement in Figure 11 (I), (II) cannot be obtained if the resisting mechanism shown in Figures 4 and 6 is assumed. If the resisting mechanism in Figures 4 and 6 is used, there should be no strain increase after reaching the plateau at lateral load – drift angle relation, say R=0.5%, since the additional force is not expected in the foundation beam. However, the increase of strain at the plateau is possible for the mechanism in Figure 9. If the rigid rotation in Figure 9 takes place, the gap opening the major shear crack at the foundation beam and the rotation angle of the wall pile assemblage should have a linear relation. It is assumed that the gap opening can be represented by integrating the strain along the longitudinal reinforcement of the foundation beam in Figure 11 (I), (II), and that the rigid rotation of the assemblage can be represented by the drift angle of the specimen. In Figure 11 (IV), the elongation of the upper longitudinal bars at different drift angle is potted. It is clear that the elongation of the reinforcement is linearly proportional to the drift angle in either direction for two specimens. With this rigid rotation mechanism, the increase of strains at the upper longitudinal reinforcement can be consistently explained even after the lateral load reached the plateau.

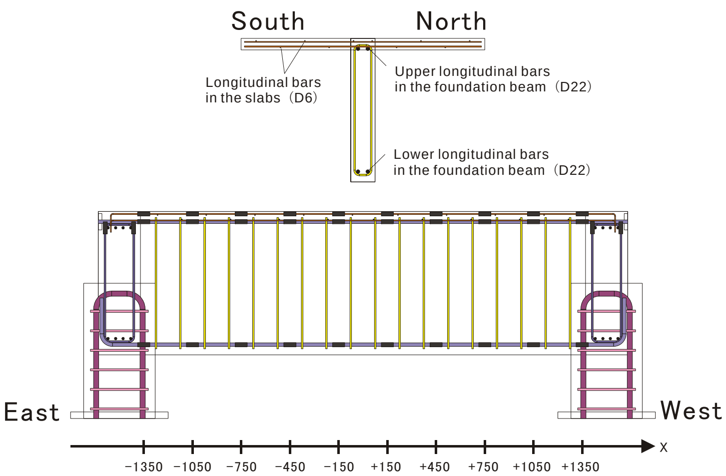
The rigid rotation mechanism implies that the upper reinforcement should be designed considering the drift angle, of the structural wall. Especially at large drift angle, the required stress or strain of the upper longitudinal reinforcement can be under estimated if the conventional mechanism in Figure 6 is used.



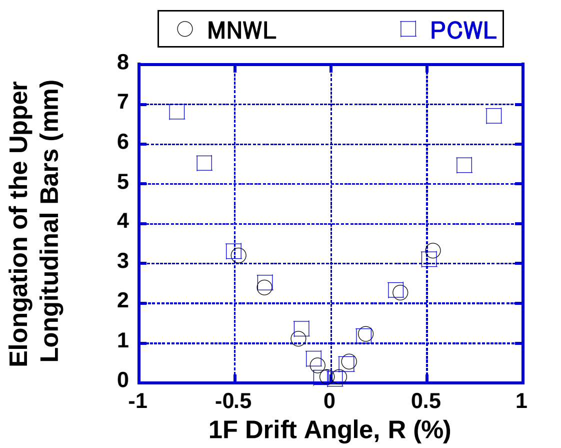
(I) MNWL



(II) PCWL



(III) Location in the foundation beam



(IV) Elongation of the upper longitudinal bars- first story drift angle, R relations

Figure 11. Strain distributions of upper longitudinal reinforcement in foundation beams

4. CONCLUSIONS

In order to investigate the lateral load resisting mechanism of the structural wall – foundation assemblage, two types of specimens were tested. One of the specimens had monolithic wall and the other specimen had precast wall which had vertical and horizontal joints around precast wall panels.

- The number of cracks in PCWL is greater than that of MNWL. Cracks across the wall panels and along the member interfaces were a major source of deformation of the structural wall and two specimens showed different deformation modes. Nevertheless, hysteresis loops of two specimens were similar with respect to stiffness, capacity and energy dissipation because the flexural deformation mode dominated the total deformation and the rocking motion of wall panel in the precast specimen was not activated. The stiffness of the surrounding beams and column was also sufficient to give enough constraint on the rocking motion.
- The conventional lateral force resisting mechanism assuming the reparation of structural walls and foundations may not be used when the shear crack penetrated the foundation beam and the structural wall – pile assemblage started to rotate rigidly. In this case, the rigid rotation model should be used to correctly simulate the stress and strain conditions of the upper the reinforcement of the foundation beam. The use of conventional mechanism under large drift angle could underestimate the stress and strain in design process and may cause the unexpected damage to the foundation beams.

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