- Technical Paper -

INVESTIGATION OF SEISMIC RETROFIT FOR PILOTIS FRAMES UTILIZING EXTREMELY THICK HYBRID WALLS

Md. Nafiur RAHMAN*1,Tetsuo YAMAKAWA*2, Yoichi MORISHITA*3 and Kozo NAKADA*4

ABSTRACT

The past earthquakes revealed that many of the pilotis buildings had suffered the extensive structural and non-structural damages. Considering this fact, two new seismic retrofit techniques utilizing extremely thick hybrid walls (namely, wing-wall, panel-wall) constructed in the first story pilotis frames are proposed in this paper. The major objectives of the proposed retrofit techniques are to enhance the lateral strength and ductility of the pilotis frames. The assessment of the proposed retrofit techniques are experimentally investigated and analytically evaluated.

Keywords: seismic retrofit, pilotis frame, thick hybrid wall, steel plate, pre-tensioned high strength steel bar, lateral force resistance capacity, ductility.

1. INTRODUCTION

The pilotis type buildings (i.e., buildings with a soft first story) are very customary in the urban areas to provide adequate open spaces for parking or good amenity through ventilation in the first story. But the investigations and observations after past earthquakes, in particular from the 1995 Hyogoken-Nanbu Earthquake in Japan revealed that many of the pilotis buildings designed with both older and updated codes had suffered the extensive structural and non-structural damages. Most of the damages were concentrated on the first story due to the abrupt change in lateral strength and stiffness. Although, the presence of various kinds of walls (namely, spandrel walls, wing-walls) inadvertently increases the lateral strength, stiffness and energy dissipation capacity of stories above the first story, this generally creates a structural vertical discontinuity of stiffness and strength which can cause the formation of so-called soft-story mechanism in the first story during earthquake.

From the past earthquake background, it is identified that the seismic vulnerability in Okinawa is still lowest in Japan. However, the pilotis type buildings although came to be recognized as a weak earthquake resistant structure, it is widely constructed in Okinawa. An investigation [1] on existing pilotis type RC multiple dwelling houses in Okinawa clarified that the seismic retrofitting is necessary to enhance the lateral strength and ductility of the 1st story pilotis frame due to the lack of sufficient seismic performance. In another investigation by T. Yamakawa et al. [2][3], it is verified that the seismic performance of column attached with secondary walls (namely, spandrel walls, wing-walls) can be improved by converting these thin secondary walls into thick walls with additional concrete sandwiched by steel plates on both sides and high strength steel bar prestressing. Moreover, in case of column with wing-wall only one side, the high seismic performance was also ensured by integrally retrofitting the column encased with steel channel in addition to grouting of cementing material into the gap within the column surface and steel plate, and the attached wing-wall with additional concrete sandwiched by steel plates and high strength steel bar prestressing [4]. Based upon these previous investigations, two kinds of retrofit techniques for the case of 1st story pilotis frames are proposed in this paper. In order to do so, two original frame specimens are retrofitted by constructing wing-walls with opening inside the frame and one more original frame is retrofitted by constructing panel wall without opening. The main goal of this research is to verify whether the retrofit techniques proposed for the pilotis frames are effective or not in respect of increasing the lateral force resistance capacity and ductility. The assessment of the proposed retrofit techniques are experimentally investigated and also analytically evaluated.

*1 Graduate Student, Graduate School of Eng. and Science, University of the Ryukyus, M. Eng., JCI Member
*2 Prof., Department of Civil Eng. & Architecture, University of the Ryukyus, Dr. Eng., JCI Member
*3 Associate Prof., Dept. of Civil Eng. & Architecture, University of the Ryukyus, Dr. Eng., JCI Member
*4 Research Associate, Dept. of Civil Eng. & Architecture, Univ. of the Ryukyus, M. Eng., JCI Member

2. TEST PLAN

In order to attain the seismic performance of pilotis buildings, it is necessary to ensure the lateral force resistance capacity of 1st story pilotis part. For this purpose, bracing, earthquake resistant wall etc. are recently used in the pilotis frames. In this research, an opening type wing-wall and non-opening type panel wall laterally reinforced by steel plates and high strength steel bar prestressing are newly established for retrofitting the pilotis frames.

In order to ascertain the effectiveness of the proposed retrofit techniques, three retrofitted pilotis frame specimens were tested under the combination of cyclic lateral forces and a constant axial load simultaneously. At first, each original frame specimen consisting of two square columns (depth & width=250mm, clear height=1,000mm) and a beam (depth=400mm, width=200mm, clear length= 1,500mm) was cast monolithically with a stab (depth=500mm, width=600mm, length=2,300mm) at bottom. About three months later, the original pilotis frame specimens were retrofitted with additional concrete sandwiched by steel plates and high strength steel bar prestressing. Then after about five to six weeks, the cyclic loading tests were carried out. The specimens R04P-O and R04P-OR were retrofitted by constructing wing-walls with opening inside the frame. But the main difference between them was that no additional reinforcement was provided inside the wing-walls of specimen R04P-O. Moreover, in case of specimen R04P-OR, 4-D16 rebars were anchored (anchorage length=130mm) through the beam at top and stab at bottom by utilizing chemical setter. The hoops (D6-@100mm) were also provided. Another test specimen R04P-W was retrofitted by constructing full panel wall without opening inside the pilotis frame and no additional reinforcement was provided in the wall

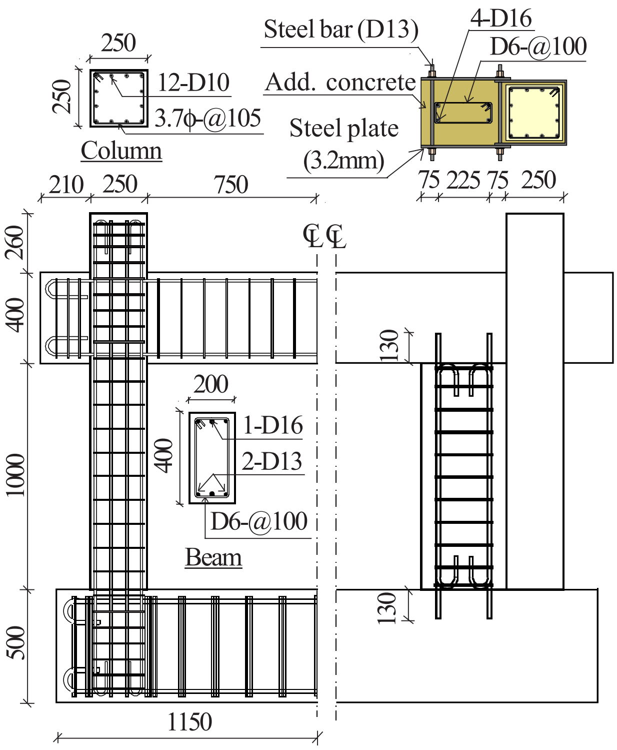


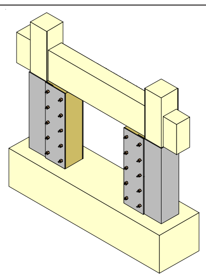
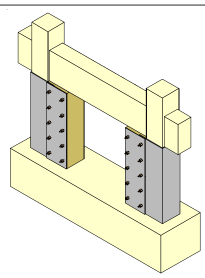
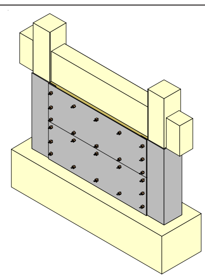
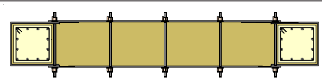
Fig. 1 Details of reinforcement (unit: mm)

Table 1 Properties of steel materials

		a (cm^2)	f_y (MPa)	ε_y (%)	E_s (GPa)
Rebar	D10	0.71	405.8	0.23	173.6
	D13	1.27	331.1	0.19	174.7
	D16	1.99	327.1	0.19	175.0
Hoop or Stirrup	3.7ϕ	0.11	560.3	0.29	191.5
	D6	0.32	443.2	0.27	164.3
Steel bar	13ϕ	1.33	1220.0	0.61	200.0
Steel plate	t=3.2 mm	-	286.0	0.12	236.0

Notes: a = cross sectional area; f_y= yield strength of steel; ε_y= yield strain of steel; E_s=Young's modulus of elasticity.

Table 2 Details of test specimens (unit: mm)

	R04P-O	R04P-OR	R04P-W
Elevation			
Cross section			
σ_B (frame)	28.1 (MPa)	28.1 (MPa)	29.7 (MPa)
Common details	Axial force ratio, $N/(bD\sigma_B)$ = 0.1 (per column); Additional concrete, $\sigma_{B(add.)}$ = 30.6 MPa; Reinf. in column:- main reinf.:12-D10(p_g=1.36%), hoop:3.7ϕ-@105 (p_w=0.08%); Reinf. in beam:-main reinf.:2-D13, 1-D16 (top & bot.) (p_g=1.14%), stirrup:D6-@100 (p_w=0.32%).		

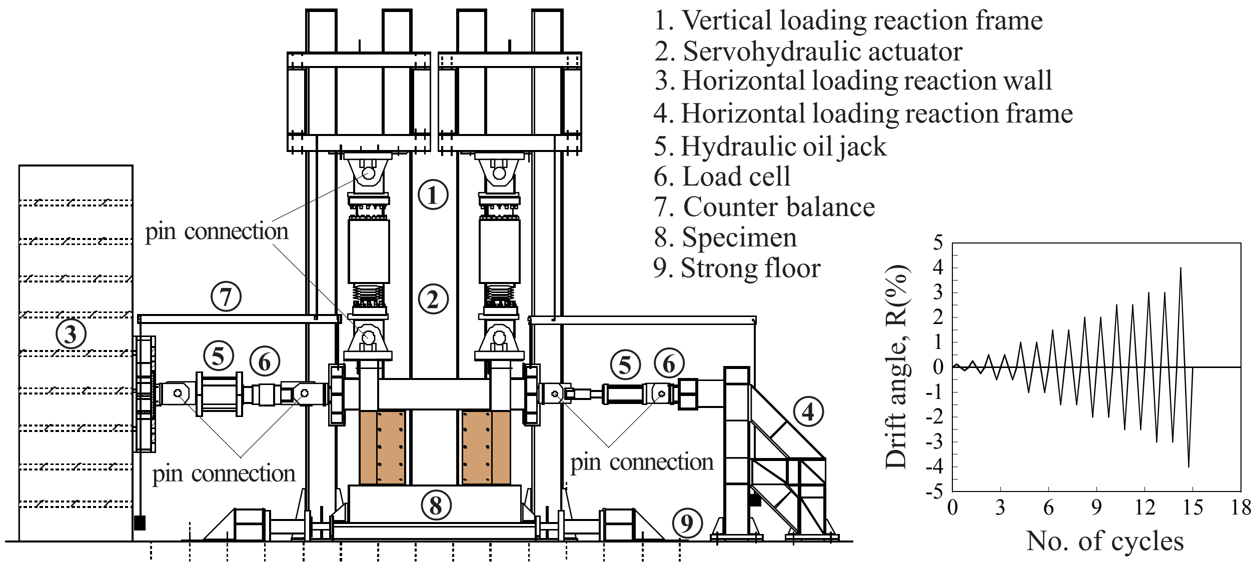


Fig. 2 Test setup and loading program

panel. In all the three retrofitted specimens, the main square column was encased with steel channel and the thin steel plates were linked with this steel channel utilizing high strength steel bars (diameter=13mm) to form a framework with opening equal to same width of column. This opening is filled up with additional concrete to make it as a wall. After hardening of post-cast concrete, pretension force was applied in the high strength steel bars that were penetrated across the wall beforehand. The level of pretension strain of steel bar was about 1,250μ (at a stress of 250 MPa). Moreover, cement slurry (compressive strength=50MPa) was grouted to eliminate the gap within the column surface and steel channel.

The mechanical properties of the steel materials employed in the test specimens are listed in Table 1. Schematic figures of retrofit techniques and details of reinforcement arrangement are presented in Table 2 and Fig. 1 respectively. The test setup and loading program are illustrated in Fig. 2. During the cyclic loading test, the axial load was applied by two vertical servohydraulic actuators with capacity of 1,000 kN each. The cyclic lateral force was applied by single acting jack system with compressive force only in both positive and negative directions. In case of specimens R04P-O and R04P-OR, the cyclic loading test was carried out in the range of drift angle ± 0.5%, ± 1.0%, ± 1.5%, ± 2.0%, ± 2.5% and ± 3.0% at two successive cycles, and ± 0.125%, ± 0.25% and ± 4.0% at one cycle. But in case of specimen R04P-W, the cyclic loading test was not continued after the drift angle of ± 2.0% due to the punching shear failure and the damage in the zone of beam-column connection. The scale factor of the specimen was about 1/2.4 to model a low-rise school building designed according to pre-1971 design code. The axial force ratio ($N/(\sigma_B bD)$) was 0.1 per square column only.

3. EXPERIMENTAL RESULTS AND DISCUSSIONS

The observed cracking patterns and damage conditions of the retrofitted specimens at final drift angle are presented in Fig. 3. Since the column and additional wing-walls or panel wall were covered by thin steel plates, the cracking patterns of these portions were detected by detaching the steel plates after the end of test. The experimental results on the relationship between the shear force V and the story drift angle R are illustrated in Fig. 4. The dotted lines drawn in the V-R curves are the calculated flexural strength of frame whose columns are virtually retrofitted (assumed retrofitted for calculation only, but cyclic loading test was not done) by corner blocks and high strength steel bar prestressing [5]. The variations of accumulated absorbed energy (W) with drift angles for the retrofitted specimens are presented in Fig. 5.

In retrofitted specimen R04P-O, first crack was generated in beam at R=0.125%. The longitudinal reinforcement started yielding in column at R=0.2% and in beam at R=0.9%. At R=1.0%, shear crack formed in beam. During the cyclic loading test, a loud sound was appeared at a drift angle of about -1.5% and one longitudinal reinforcement was fractured at the bottom end of column. The flexural crack was also observed at the bottom of column and wing-wall which indicated the formation of plastic hinges at that location. Since the column with additional wing-wall was united firmly, the rigid body rotation appeared within the formed plastic hinges. Moreover, during the cyclic loading, the beam was subjected to a remarkable axial force, and due to this axial force, the flexural strength of beam increased and it might exceed the shear and bond strength. Therefore, with the increase of drift angle, the damage on beam due

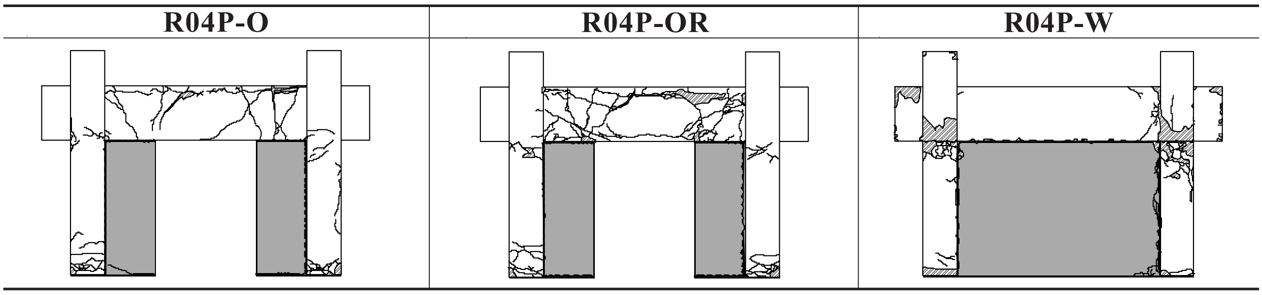


Fig. 3 Observed cracking patterns at final drift angle

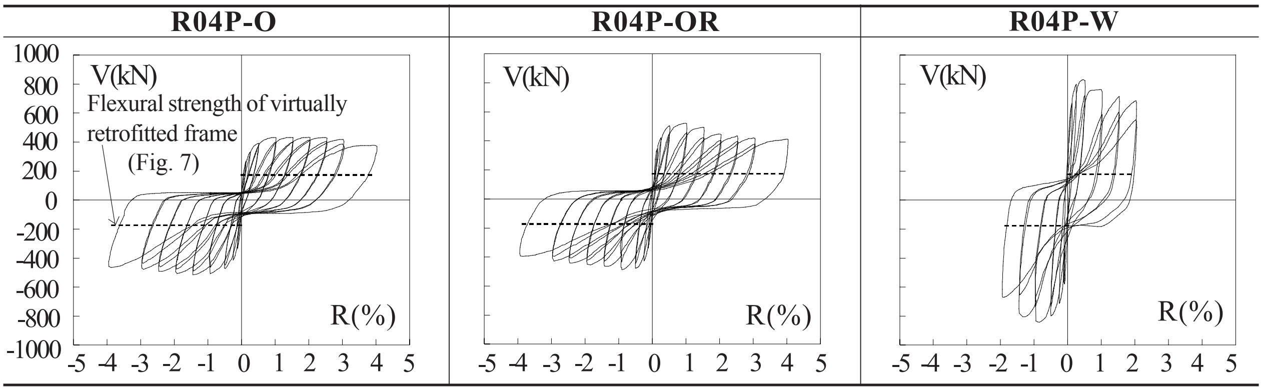


Fig. 4 Experimental *V-R* relationships

to shear and bond occurred progressively. However, the lateral force resistance capacity maintained perfectly even at large drift angles as shown in Fig. 4. The cyclic loading test was stopped at R=4% with one cycle and did not continue further due to the large damage in beam.

Since in test specimen R04P-OR, the additional reinforcement as 4-D16 longitudinal rebar and D6-@100mm hoops were provided in additional wing-walls, the lateral force resistance capacity of this specimen increased to about 20% than that of R04P-O as shown in Fig. 4. However, after the drift angle of 1.5%, the lateral force resistance capacity decreased gradually due to the cone-type failure of concrete in the anchorage zone. Therefore, it is understood that the anchorage is not effective at large drift angles. Moreover, the damage in beam was also larger than that of R04P-O. However, in this specimen, flexural crack was also observed at the bottom of column and wing-wall, and the rigid body rotation appeared too.

In case of test specimen R04P-W retrofitted by non-opening type panel wall without additional reinforcement inside the wall, the lateral force resistance capacity increased significantly. But, at a drift angle right after 0.5%, the capacity decreased gradually. Since the panel wall was cast separately and no additional anchorage between the beam and panel wall, there was weak bonding at that location. Therefore, during the cyclic loading, although flexural crack was appeared first at bottom of column. But, immediately after, the punching shear crack was generated at the top of column near the

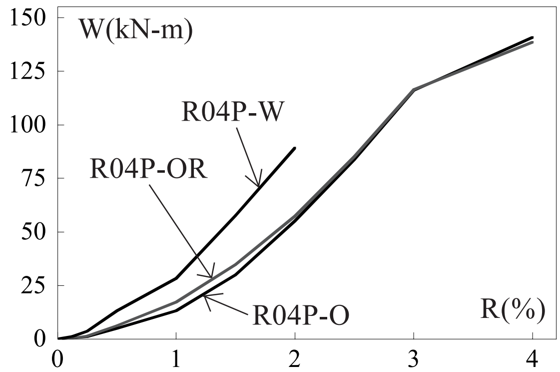


Fig. 5 Accumulated absorbed energy

beam-column connection and hence, the lateral force resistance capacity decreased gradually. For the sake of safety, the cyclic loading test was not continued after R=2.0%. Therefore, to maintain the ductility at large drift angles, the punching shear failure should be protected.

From *W-R* curve in Fig. 5, it is observed that in case of specimen R04P-OR, the accumulated absorbed energy within R=2% is slightly larger than that of R04P-O, but after this drift angle, it is almost same. Therefore, in the context of energy absorption, it may be concluded that the anchorage of additional reinforcement in specimen R04P-OR is not so much effective at large drift angles. However, in case of specimen R04P-W retrofitted by non-opening type panel wall, the accumulated absorbed energy increased significantly than the specimens R04P-O and R04P-OR retrofitted by opening type wing-walls. In Fig. 5, it is also observed that in specimens R04P-O and R04P-OR, there is noticeable change in slope. Because after this drift angle, the cyclic loading test was carried out for one cycle each instead of two cycles.

4. ANALYTICAL INVESTIGATION

In order to suggest the design guidelines of the retrofit techniques proposed for pilotis frames, the analytical investigations are carried out to conform with the experimental results. The shear and flexural strengths of beam and column of the original frame are calculated by AIJ simplified equations [6]. In order to compare the experimental lateral force resistance capacity of the retrofitted pilotis frames, two virtual pilotis frames are also analyzed. Among them, one is non-retrofitted frame in which shear failure happens. The other one is flexural failure type in which the columns were virtually retrofitted by utilizing corner blocks and high strength steel bar prestressing.

The lateral force resistance capacity for specimens R04P-O and R04P-OR as well as virtual specimens are calculated based on the mechanism of plastic hinge formation. In this calculation, the beam-column connection is considered as rigid and the centerline dimension of the frame is taken into account. The flexural strength of column retrofitted with additional wing-wall is calculated by considering as a unified section. Moreover, since the strength of additional concrete is nearly equal to that of the original frame, for simplicity, the concrete strength for unified section is considered as same as original frame. In case of specimens R04P-O and R04P-OR, the unified column with wing-wall section is asymmetric about the center line of square column section. Therefore, the section has two different moment capacity depending on the situation of compression or tension either in column side or in wall side during the cyclic loading. The flexural strength of this unified section is calculated more accurately by fiber model. The axial force(N)-moment(M) interaction diagrams for square column section only calculated by AIJ simplified equation and for unified wing-wall column section calculated by fiber model are shown in Fig. 6. The experimental skeleton curves and the calculated results for different specimens are presented in Fig. 7.

From Fig. 7, it is observed that although the calculated lateral force resistance capacity of virtually retrofitted pilotis frame is increased a little but the failure mode can be possible to shift from shear to flexural one. In such case, the ductility may also be enhanced. Again, in retrofitted specimen R04P-O without additional reinforcement in wing-wall, the experimental lateral force resistance capacity (431.6 kN) was increased to about 2.5 times the capacity of non-retrofitted pilotis frame (167.9 kN) and also maintained even at large drift angles. On the other hand, in retrofitted specimen R04P-OR with additional reinforcement in wing-wall, the experimental lateral force resistance capacity (522.6 kN) was increased to about 3 times the capacity of non-retrofitted pilotis frame and also maintained until about R=1.5%, but afterwards decreased gradually with the increase of drift angle due to the cone-type failure of concrete in the anchorage zone. In both the cases, the calculated lateral force resistance capacity of the retrofitted specimens agreed well with the test results.

In case of specimen R04P-W retrofitted with non-opening type panel wall and without additional reinforcement inside the wall, the experimental lateral force resistance capacity (829.4 kN) was increased to about 5 times the capacity of non-retrofitted pilotis frame. The calculated flexural strength [7] of this specimen also

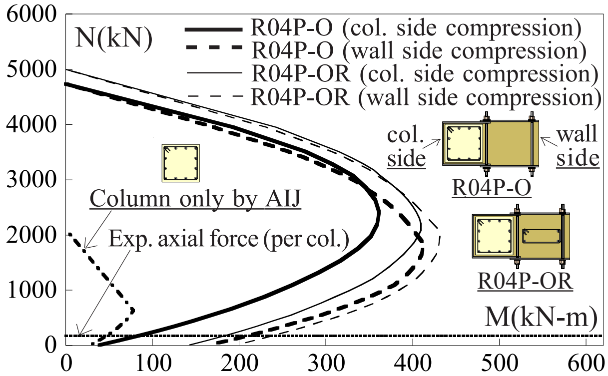


Fig. 6 *N-M* interaction diagrams by fiber model

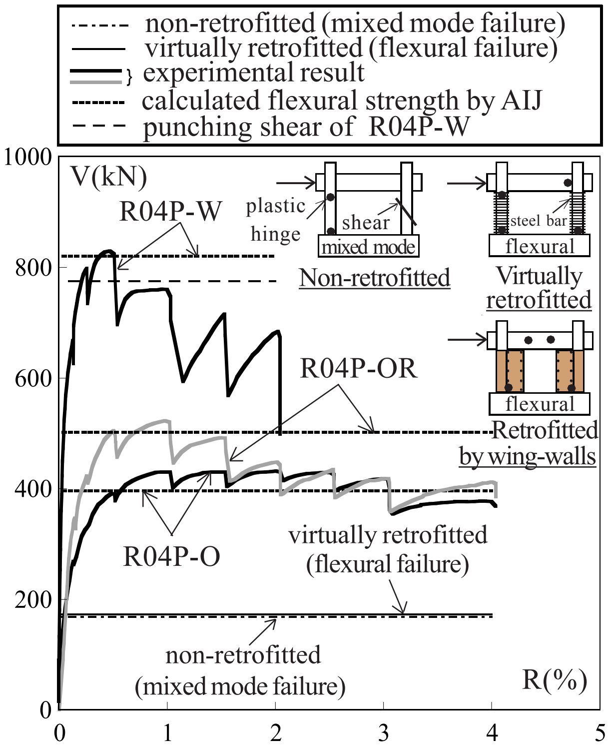


Fig. 7 Comparison of test and calculated results

agreed well with the experimental result. However, after the drift angle of 0.5%, the experimental lateral force resistance capacity of this specimen decreased gradually due to the punching shear failure at the beam-column junction through the beam-panel wall connection line. Since the panel wall was cast separately and no additional anchorage between the beam and panel wall, there was weak bonding at that location. Therefore, to maintain the ductility at large drift angles, the beam-panel wall connection can be strengthen by providing stud dowel at that location. For this specimen, the punching shear is calculated as the summation of punching shear of concrete and rebars of columns. The punching shear strength of concrete is calculated according to ACI 318-99 design code [8]. The shear strength of rebar is calculated as $F_y/\sqrt{3}$, where, F_y is the yield strength of rebar.

5. CONCLUSIONS

(1) The retrofit techniques of R04P-O and R04P-OR in which the pilotis frames are retrofitted by casting additional wing-walls with or without additional reinforcement inside the wall and sandwiched by steel plates and high strength steel bar prestressing endorse the effective improvement of seismic performance.

(2) Although in specimen R04P-OR in which the additional reinforcement is provided inside the wing-walls, the lateral force resistance capacity is initially increased , but the ductility is not maintained with the increase of drift angle due to the cone-type damage of concrete in the anchorage zone. Therefore, in such case, it seems that the anchorage with additional reinforcement is not so much effective.

(3) Experimentally, it was observed that in specimens R04P-O and R04P-OR, remarkable damages occurred in beams. Because, during the cyclic loading, the beam was subjected to a significant axial force, and due to this axial force, the flexural strength of beam relatively increased and it might nearly equal or exceed the shear and bond strength.

(4) In case of specimen R04P-W retrofitted by non-opening type panel wall without additional reinforcement inside the panel wall, the high lateral force resistance capacity is achieved, but the ductility is not ensured due to the punching shear failure in top of columns through the junction line between the beam and panel wall. Therefore, to ensure the ductility at large drift angles with preventing the punching shear failure, stud dowel can be provided in the location of beam-panel wall and panel wall-bottom stub connections.

(5) Finally, it may be concluded that the seismic performance of pilotis type RC buildings can be improved with the enhancement of both lateral force resistance capacity and ductility of the first story pilotis frame by retrofitting with opening type extremely thick hybrid walls.

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