EXPERIMENTAL AND ANALYTICAL INVESTIGATION OF SEISMIC RETROFIT FOR RC FRAMED SHEAR WALLS

S. Takara[1], T. Yamakawa[1], K. Yamashiro[1]

[1]University of the Ryukyus, Okinawa, Japan
E-mail: takara@kne.biglobe.ne.jp

ABSTRACT :

The RC framed shear walls are popular to resist the earthquake loads efficiently. The recent past earthquakes reveal that they are vulnerable to large seismic excitation. To satisfy the demand of high energy absorption by large earthquake, it is necessary to retrofit the existing framed shear walls with converting the failure mode from shear to flexural or uplift one. Considering these facts, two retrofit techniques are proposed here. One is retrofitting of column only using steel plates, steel corner blocks and high strength steel bar (referred to as PC bar hereafter) prestressing. The other one is retrofitting of framed shear wall using additional concrete sandwiched by steel plates and PC bar prestressing. The proposed techniques are experimentally investigated and analytically evaluated. The seismic performance of the specimens are also discussed here.

KEYWORDS: *Seismic retrofit, Framed shear wall, High strength steel bar, Lateral capacity, Ductility*

1. INTRODUCTION

The RC framed shear walls are very popular in Japan as they can be very efficient in resisting the earthquake loads. However, the observations and investigations from recent past earthquakes reveal that they are vulnerable to large seismic excitation. The presence of panel wall in frame increases the seismic input force and if the shear strength is insufficient, the undesirable brittle shear failure is likely to happen by forming a large diagonal crack throughout both the panel wall and frame. Therefore, to satisfy the demand of high energy absorption by large earthquake, it is necessary to retrofit the existing framed shear wall with converting the failure mode from shear to flexural or uplift one by increasing the shear strength only or both the shear strength and ductility.

From the investigation by T.Yamakawa et al. [1], it is demonstrated that the seismic performance of RC columns can be improved obviously by retrofitting with PC bar prestressing as external hoops. From other investigations by T.Yamakawa et al. [2-3], it is also verified that the seismic performance of column attached with secondary 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. Based upon the previous investigation, two retrofit techniques are proposed here. One is retrofitting of column only using steel plates, steel corner blocks and high strength steel bar prestressing. The major objectives of the proposed retrofit techniques are to enhance the lateral strength and ductility of the framed shear wall. The assessment of the proposed retrofit techniques are experimentally investigated and also analytically evaluated.

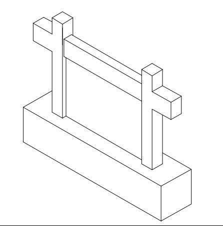
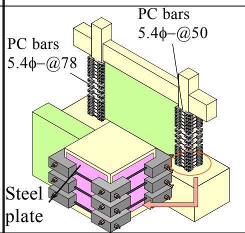
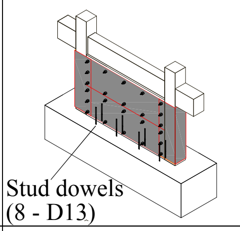
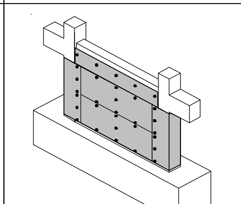
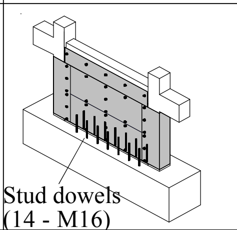
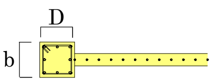
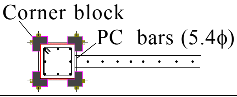
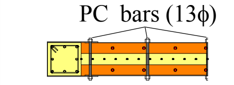
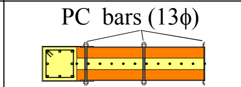
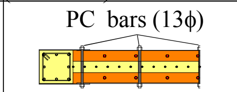
2. TEST PLAN

In order to ensure the effectiveness of the proposed retrofit techniques, one standard shear wall specimen and four retrofitted ones were tested under the combination of cyclic lateral forces and a constant axial load simultaneously. The reinforcement details of framed shear walls are presented in Fig. 1. The details of test specimens and the mechanical properties of steel materials are shown in Table 1 and Table 2 respectively. The axial force ratio ($N/(bD\sigma_B)$) was 0.2 per square column only. The test setup and the loading program are illustrated in Fig. 2. The axial load was applied by two vertical servo hydraulic actuators with a capacity of 1,000 kN each, and the cyclic lateral force was applied by double acting jack system. The cyclic loading test was carried out in the range of displacements ± 0.125mm, 0.25mm, ±4.0mm and ±5.0mm at one cycle; ±0.5mm, ±1.0mm, ±1.5mm, ±2.0mm, ±2.5mm, and ±3.0mm at two successive cycles.

The specimen R05W-P0 was non-retrofitted one. In case of R05W-P50S, only columns were retrofitted with PC bar (diameter = 5.4mm), steel plate (thickness = 2.3mm) and steel corner blocks. Three sides of the column except the side attached to the panel wall were covered by steel plates. For the top and bottom part of the column, the interval of PC bar was 50mm, and the interval for the middle part was 78mm. Before the test, the holes in the panel wall were made to insert the PC bars according to the design arrangement. The initial tension force was also applied in these PC bars. The level of pretension strain in PC bars was about 2500μ (at a stress of about 500MPa).

The specimens R05W-WR, R07W-W and R07W-S were retrofitted with casting additional concrete (up to the width of the column) sandwiched by steel plates and PC bar prestressing. The both columns were also jacketed with channel-shaped steel plates (thickness = 2.3mm). Cement slurry was grouted to eliminate the gap within the

Table 1 Details of test speimens(unit:mm)

	R05W-P0	R05W-P50S	R05W-WR	R07W-W	R07W-S
Specimen					
Cross section					
σ_B(Original)	27.8 (MPa)	27.8 (MPa)	29.5 (MPa)	24.3 (MPa)	24.1 (MPa)
σ_B(Additional)			32.3 (MPa)	25.7 (MPa)	
Common details	Axial force ratio : $N/(bD\sigma_B)$=0.2 (per column), Reinf. in column : main reinf. : 8-D10 (p_g=1.85%), hoop : 3.7ϕ-@105 (p_w=0.12%); Reinf. in beam : main reinf. : 4-D13 (p_t=0.81%), stirrup : D6-@120 (p_w=0.43%) ; Reinf. in panel wall : both horizontal and vertical : 3.7ϕ-@60 single(p_s=0.31%); panel wall thickness: t=60mm				

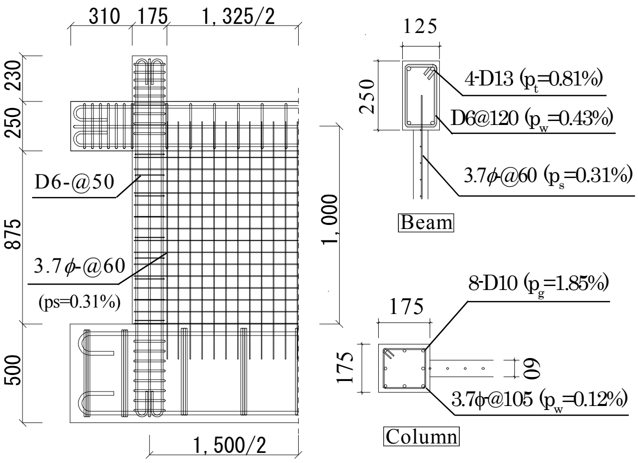


Fig. 1 Details of reinforcements (unit:mm)

Table 2 Properties of steel materials

		a (mm²)	σ_y (MPa)	σ_u (MPa)	ε_y (%)	E_s (GPa)
D10	R05W Series	71	412	577	0.21	195
	R07W Series	71	355	498	0.17	201
D13	R05W Series	127	340	524	0.17	200
	R07W Series	127	342	525	0.17	201
3.7ϕ	R05W Series	11	643	687	0.32	199
	R07W Series	11	683	735	0.31	202
D6	R05W Series	32	393	488	0.22	176
	R07W Series	32	504	585	0.24	194
Steel plate t=2.3mm	R05W Series	-	286	342	0.12	236
	R07W Series	-	331	465	0.13	221
PC bar 13ϕ	R05W Series	-	1220	-	0.61	200
	R07W Series					
dowel M16	R05W Series	-	-	-	-	-
	R07W Series	153	245	410	-	-

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

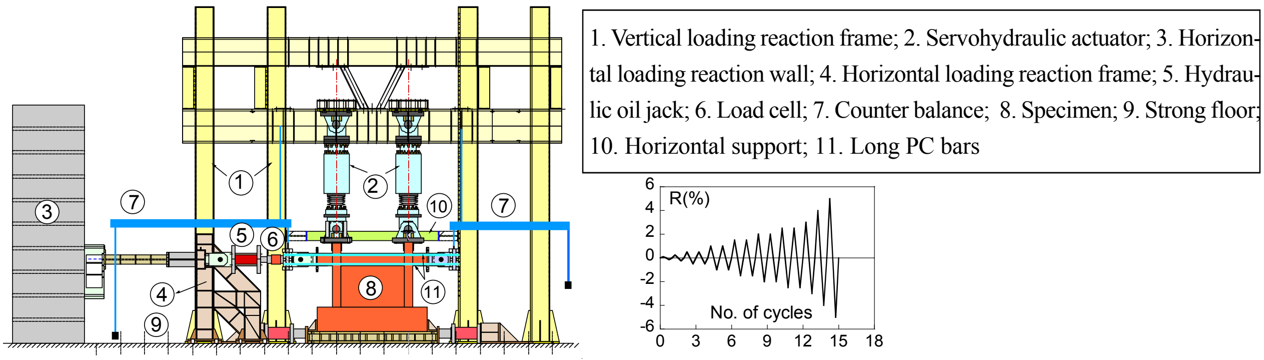


Fig. 2 Test setup and loading program

column surface and the steel channel. After the hardening of post-cast concrete, initial force was applied in the steel bars that were inserted across the wall beforehand. The level of initial tension strain of PC bar was about 1250μ (at a stress of about 250MPa). In specimen R05W-WR , stud dowels (8-D13) were used at the panel wall-stub connection (bottom only). Again in specimen R07W-W, no stud dowels were provided, but the beam-panel wall connection at top was strengthened by casting the additional concrete sandwiched by steel palates and PC bar prestressing up to the beam, no stud dowels, but it placed to the center from the bottom to the beam with the board and it crowded, it tightened with PC bars. In specimen R07W-S, stud dowels (14-M16 (SS400)) were provided at the panel wall-stub connection (bottom only), and the top connection was strengthened like a specimen R07W-W. However, vinyl tape was wrapped around the stud dowels to cut the bond between concrete and the bar.

3. EXPERIMENTAL RESULTS AND DISCUSSIONS

The experimental results on the relationship between the shear force V and the story drift δ, and the observed cracking patterns at the final drift angle are illustrated in Fig. 3. The cracking patterns were detected by detaching the steel plates after finishing the tests. The variations of accumulated absorbed energy (W) with drift of the test specimens are presented in Fig. 4.

In non-retrofitted standard specimen R05W-P0, shear crack appeared in the panel wall at δ=1.25mm (R=0.125%). This shear crack in the panel wall widened greatly and passed through the columns at δ=5mm (R=0.5%). At δ=10mm (R=1.0%) in the push (+) direction at the first cycle (cyclic load from left to right is push and vice versa), the width of shear crack in the panel wall and column was about 20mm. Then in the pull (-) direction at the same drift at δ=-6.5mm (R=0.65%), the shear failure happened and the cyclic loading test had been stopped.

In specimen R05W-P50S, between the drift of δ=5mm and δ=10mm the shear crack with a width of about 3mm was appeared in the panel wall, which caused the lateral capacity to decrease obviously. In the meantime, the flexural crack was generated at the top of the left column and bottom of the right column. After that, large damage was appeared at the top of the left column and the beam at a drift of 15mm and 30mm respectively. After finishing the test and detaching the steel plates on the column, it was observed that although the shear failure occurred in the panel wall, the shear crack did not pass through the columns since the column had been retrofitted with PC bar prestressing, steel plates and corner blocks. In this case, the lateral capacity was almost the same as that of the non-retrofitted shear wall. However, the shear failure of columns was shifted to flexural failure since the columns were retrofitted. After the appearance of shear failure in panel wall, this framed shear wall could be considered as a frame with secondary spandrel wall, and with the increase of drift , this was finally shifted to a pure frame. Although the lateral capacity decreased sharply due to the shear failure that appeared in the panel wall, the frame could still maintain the lateral capacity of pure frame until the drift of 50mm, and could sustain the vertical load.

In specimen R05W-WR retrofitted with additional stud dowels at the panel wall-stub connection (bottom only), the

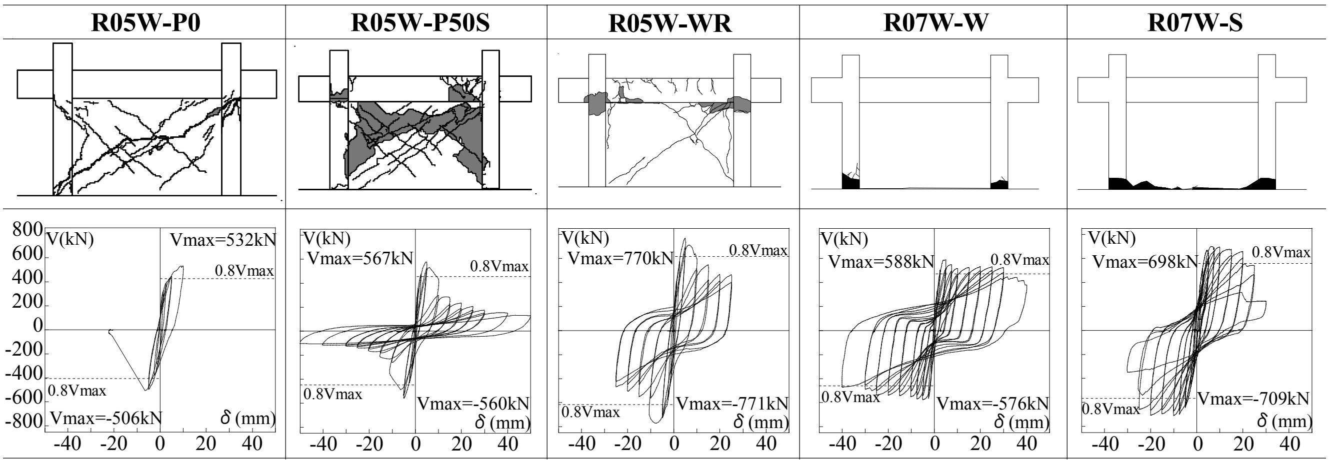


Fig. 3 Experimental V-R relationships and crack patterns

punching shear sliding initiated at δ=10mm at the beam-column connection through the beam-panel wall boundary line. Here, the stud dowels increased the flexural and punching shear strength of bottom section. Moreover, the shear sliding resistance at the compressive zone through the stub-panel wall boundary line would be supported by the frictional resistance caused by the imposed vertical loads and the overturning moment. Consequently, the punching shear strength at bottom was larger than that at top. Therefore, the punching shear sliding generated at top than bottom instead of flexural yielding at bottom. At δ=25mm, the crack width at the top of panel wall was about 20mm. In addition, the large damage was generated in beam, and the whole beam appeared in the form of an arch shape. For safety, the cyclic loading test had been stopped after δ=25mm. Here, the lateral capacity was also increased significantly and it was as large as about 1.5 times the capacity of non-retrofitted shear wall; but after δ=10mm, the capacity decreased owing to the punching shear sliding through the top beam-panel wall connection.

In specimen R07W-W in which the top beam-panel wall connection was strengthened by casting additional concrete sandwiched by steel plates with PC bar prestressing up to the beam, the longitudinal rebar at the bottom of column started yielding at δ=2.5mm. However, the experimental capacity did not reach to the calculated flexural strength due to the shear sliding though the panel wall-stab boundary line at the bottom. Since no stud dowel was provided at that location to prevent the shear sliding. However, 80% or more of the lateral capacity maintained until the displacement of 30mm. During the test, the expansion of steel plate on the bottom of the column was observed at large displacement and consequently the lateral capacity was also decreased. At δ=40mm, the decrease in the lateral capacity was clearly distinguished due to the broken of longitudinal rebar at bottom of column.

In specimen R07W-S, which was retrofitted as R07W-W except in which the additional stud dowels were provided along the panel wall-stub connection (bottom only), the experimental lateral capacity reached to the flexural strength. Because it was able to prevent sliding to some degree by stud dowels at panel wall-stub connection. In this specimen with the increase of displacement, 80% or more of the lateral capacity maintained until the δ=19.5mm. Moreover, the flexural strength decreased remarkably due to the successive breakage of rebars in column.

The variation of accumulated absorbed energy with the increase of displacement is shown in Fig. 4. It is observed that the accumulated absorbed energy in specimen R05W-P50S increased than the non-retrofitted specimen R05W-P0, the lateral capacity of frame maintained until large displacement after the broken of shear wall as the columns were retrofitted by PC bar prestressing. It is also observed that at small drift, the energy absorption in specimen R05W-WR is higher than other retrofitted specimens. Because, the lateral capacity in specimen R05W-WR was higher than the others due to the higher concrete strength and the contribution of stud dowels to flexural strength. Again, since the experimental lateral capacity of specimen R07W-W did not reached to the flexural strength due to sliding shear failure at bottom section, the accumulated absorbed energy of this specimen at small drift was smaller than that of the specimen R07W-S.

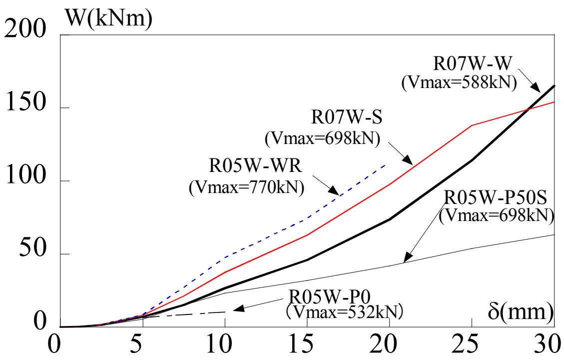


Fig. 4 Accumulated absorbed energy

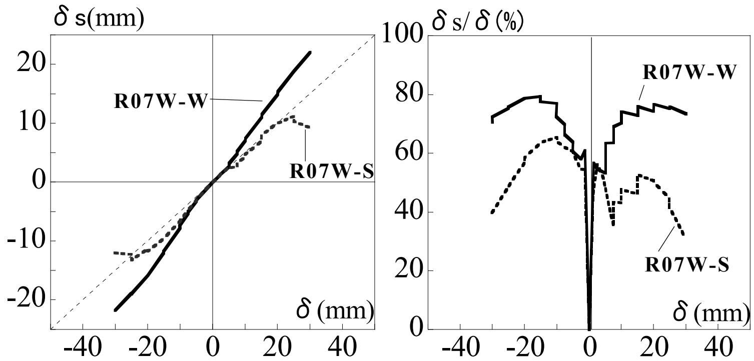


Fig. 5 Horizonal displacement and sliding displacement

The sliding displacement (δs) was measured in specimens R07W-W and R07W-S. The measurement transducer was placed at the center of the bottom of the panel wall. The results are presented in Fig. 5. The horizontal axis in the graph represents the total horizontal displacement δ, and the vertical axis represents the amount of sliding displacement (δs) (left figure). The thin dashed line in the left graph shows that the proportion of sliding with the lateral displacement is 50%. The right graph shows the percentage of sliding displacement in the total displacement. The sliding displacement in R07W-W was about 60% of the total lateral displacement until the rebar in column yielded at a lateral displacement of 2.5mm. After the yielding rebar in column, the percentage of the sliding displacement increased. However, the lateral capacity decreased when the lateral displacement exceeded 20mm, and the percentage of sliding decreased.

The sliding displacement was not able to be stopped completely even in specimen R07W-S**,** where stud dowels were arranged sufficiently. The sliding displacement of about 40% was observed at a lateral displacement of about 6.4mm when the lateral capacity reached to the maximum value and the longitudinal rebar started to yielding. In the pull (-) direction, the experimental lateral capacity reached to the maximum at a lateral displacement of 10.0mm, which is larger than that in the push direction (+) due to the large sliding displacement in the pull direction.

4. ANALYTICAL INVESTIGATIONS

The flexural strength of framed shear wall is calculated by Eqn. 1 based on the JBDPA guidelines [4]. Fig. 6 (a) shows the comparison of axial force (N)-moment (M) interaction diagram calculated by fiber model before and after retrofitting of the specimen R07W-S in which the experimental lateral capacity was governed by the flexural strength. It is observed that for a axial force ratio up to 0.33 ($0.5N/(bD\sigma_B)$), the flexural strength is almost the same before and after retrofitting. This is also verified through the experiment. Fig. 6 (b) shows the comparison of N-M interaction diagrams calculated by the fiber model and the simplified Eqn.1 for a specimen R07W-S. It is observed the test result agrees well with both the calculated results. It is also observed that the calculated flexural strengths by the simplified equation fiber model are almost the same at the axial force ratio between 0.1 to 0.3. Fig. 6 (c) shows the variation of the neutral axis depth with the axial force ratio before and after retrofitting the specimen R07W-S. From this figure, it is verified that during the test with an axial force ratio of 0.2, the neutral axis remains within the column. It is also observed that for an axial force ratio up to 0.21, the additional concrete has no inference in the flexural strength. Therefore, up to this axial force ratio, the simplified equation can be well applied.

The shear strength of the shear wall (${}_wQ_{su}$) is calculated base on the Arakawa mean equation [4], with also considering the steel plate as hoop. In the original equation, the limit of transverse reinforcement ratio (${}_wp_s$) is less than or equal to 0.012. When the steel plate is considered as hoop, the ratio (${}_wp_s + {}_sp_s$) considering the steel plate ratio (${}_sp_s$) also satisfies the limiting ratio of 0.012. Since the variation of the strength of additional concrete and the original shear wall is small, for simplicity, the concrete strength of the original shear wall is considered for the unified retrofitted section.

The punching shear strength is calculated by Eqn. 3 based on the JBDPA guidelines [4]. The shear strength of the panel wall (${}_{w}Q'_{su}$) is added to the punching shear strength ($2{}_{c}Q_{pu}$) of both columns. Since in specimens, **R07W-W** and **R07W-S**, the top beam-panel wall connection was strengthened by casting additional concrete sandwiched by steel plates and PC bar prestressing up to the beam, the contribution of steel plate is also considered in the calculation of punching shear strength. The shear strength of steel plate may not reach to its maximum value due to the possibility of warping or movement of steel plate during the loading test. Considering this fact, the shear strength of steel plate is decreased to 30% (α_s=0.3). In addition, when the stud dowels are arranged at the bottom of the panel wall, the shear strength of stud dowels (${}_{a}Q_{su}$) is also considered base on the JBDPA guidelines [5]. Therefore, to calculate the punching shear strength (${}_{w}Q_{pu}$) of a specimen, the minimum between the shear strength of steel plate (${}_{s}Q_{su}$) and the shear strength of anchor (${}_{a}Q_{su}$) is added to ($2{}_{c}Q_{pu}+{}_{w}Q'_{su}$) as shown in Eqn. 3. The calculated result by this equation agreed well with the test results, so this equation can be well applied.

The comparison of test and calculated results are shown in Fig. 7. In specimen R05W-WR, although the experimental lateral force capacity exceeded the calculated flexure strength (744kN without stud dowels), however the punching shear (721kN) failure was observed through the panel wall-beam connection at top. Since the stud dowels used in this specimen were bonded, they seemed to contribute in the increase of experimental flexural strength.

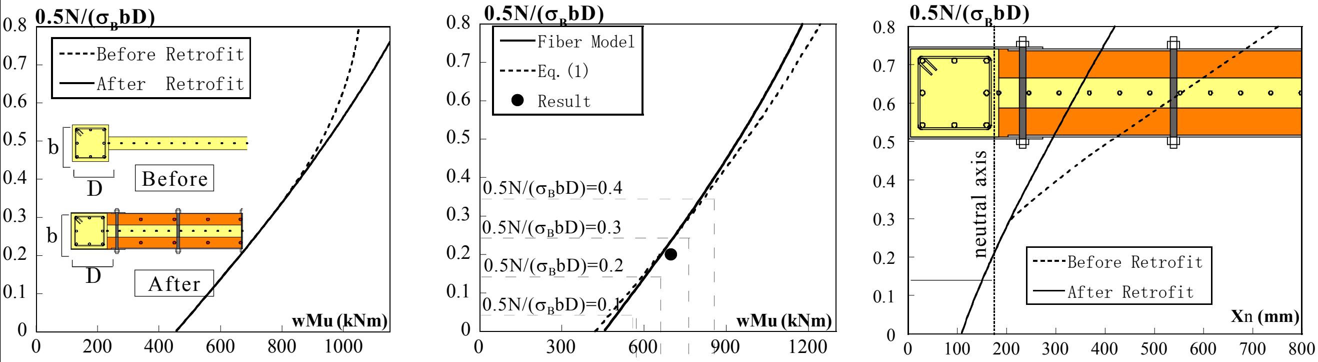
In case of specimen R07W-W in which the connection between the beam and the panel wall at top was strengthened up to the beam by casting additional concrete sandwiched by steel plates and PC bar prestressing, the experimental lateral capacity (588kN) did not reach to the calculated flexural strength (655kN). Moreover, the experimental lateral capacity of this specimen is near to the punching shear strength (618kN), which is governed by the bottom panel wall connection. Therefore, the experimental capacity did not reach to the maximum value due to shear sliding. However, the experimental capacity maintained until large displacement.

Again, in specimen R07W-S, since the calculated punching shear strength (868kN) was sufficient, the experimental lateral force capacity (698kN) reached to the calculated flexural strength (653kN). However, some sliding displacement was also observed. So, the sufficient stud dowels can not prevent the sliding displacement completely, but they can ensure the flexural strength.

Flexural strength

$$ {}_{w}M_u = a_g \sigma_y L_w + 0.5\Sigma(a_w\,{}_{w}\sigma_y)L_w + 0.5NL_w \quad (1)$$

${}_{w}M_u$:ultimate moment capacity, N:total axial load, L_w:length between columns, a_g:total area of longitudinal rebars at the tention side of column, σ_y:yield strength of rebars in column, a_w:total area of longitudinal rebars in panel wall, ${}_{w}\sigma_y$:yield strength of rebars in panel wall



(a) N-M intreraction diagram (before and after retrofit)

(b) N-M interaction diagrams by fiber model and simplified Eqn. 1

(c) Variation of neutral axis with axial force

Fig. 6 Comparisons of N-M interaction diagrams calculated by fiber model and the proposed simplified equation

Shear strength

$$ {}_{w}Q_{su} = \left(\frac{0.068\, p_t^{0.23} (18 + \sigma_B)}{\sqrt{M / (QD_w) + 0.12}} + 0.85 \sqrt{{}_{w}p_s \cdot {}_{w}\sigma_y + {}_{s}p_s \cdot {}_{s}\sigma_y} + 0.1\sigma_0 \right) b \cdot j \tag{2} $$

$M/(QD_w)=1$ when $M/(QD_w) \leqq 1$, $M/(QD_w)=3$ when $M/(QD_w) \geqq 3$

${}_{w}Q_{su}$:ultimate shear strength, b:width of column, $j=7/8(D_w-D/2)$, D_w:depth of specimen, D:depth of column, p_t:ratio of tention rebers in column $p_t=100a_t/(bD_w)$, a_t:total area of longitudinal rebars at the tention side of column, σ_B:cylinder strength of concrete of existing shear wall (MPa), $\sigma_0=N/(bD_w)$, ${}_{w}p_s$:ratio of rebars in panel wall, ${}_{w}\sigma_y$:yield strength of rebars in panel wall, ${}_{s}p_s$:ratio of steel plates in panel wall ${}_{s}p_s=2t_s/b$, ${}_{s}\sigma_y$:yield strength of steel plate, t_s:thickness of steel plate, However, ${}_{w}p_s + {}_{s}p_s \leqq 0.012$

Punching shear strength

$$ {}_{w}Q_{pu} = 2\,{}_{c}Q_{pu} \text{(punching shear strength of both column)} + {}_{w}Q'_{su} \text{(shear strength of panel wall)} $$
$$ + \min\left(2\,{}_{s}Q'_{su} \text{(shear strength of steel plates) or } {}_{a}Q_{su} \text{(shear strength of stud dowels)} \right) \tag{3} $$

$$ {}_{c}Q_{pu} = 0.167 \cdot \tau_0 \cdot b \cdot D \tag{4} $$

$$ {}_{w}Q'_{su} = \max\left({}_{w}p_s\,{}_{w}\sigma_y \text{ or } \sigma_B/20 + 0.5\,{}_{w}p_s\,{}_{w}\sigma_y \right) t_w l_w' \tag{5} $$

$$ {}_{s}Q'_{su} = \alpha_s t_s l_w'\,{}_{s}\sigma_y / \sqrt{3} \tag{6} $$

$$ {}_{a}Q_{su} = \min\left(0.7\,{}_{a}\sigma_y \cdot \Sigma_s a_e \text{ or } 0.4\sqrt{(E_c \cdot \sigma_B)} \cdot {}_{s}a_e \right) \tag{7} $$

$$ \begin{aligned} \tau_0 &= 0.98 + 0.1\sigma_B + 0.85\,\sigma \quad (0 \leqq \sigma \leqq 0.33\,\sigma_B - 2.75) \\ &= 0.22\,\sigma_B + 0.49\sigma \quad (0.33\,\sigma_B - 2.75 < \sigma \leqq 0.66\,\sigma_B) \\ &= 0.66\,\sigma_B \quad (\sigma > 0.66\sigma_B) \end{aligned} \tag{8} $$

$$ \sigma = p_g \sigma_y + 0.5N / (bD) \tag{9} $$

${}_{w}Q_{pu}$:ultimate punching shear strength of specimen, ${}_{c}Q_{pu}$:ultimate punching shear strength of column, ${}_{w}Q'_{su}$:shear strength of panel wall , ${}_{s}Q'_{su}$:shear strength of steel plates, ${}_{a}Q_{su}$:shear strength of stud dowels, b:width of column, D:Depth of column, σ_B:cylinder strength of concrete of existing shear wall (MPa), t_w:thickness of panel wall of existing shear wall, l_w':length of panel wall , ${}_{w}p_s$:ratio of rebars in panel wall, ${}_{w}p_s=\Sigma a_w / (t_w l_w')$, a_w:total area of longitudinal rebars in panel wall, ${}_{w}\sigma_y$:yield strength of rebars in panel wall, p_g:ratio of total area of longitudinal rebars in panel wall, σ_y:yield strength of rebars in column, t_s:thickness of steel plates, t_w:thickness of panel walls, ${}_{s}\sigma_y$:yield strength of steel plates, α_s:reduction factar for shear strength of steel pletes (α_s=0.3), N:total axial load, ${}_{a}a_e$:total area of stud dowels, ${}_{a}\sigma_y$:yield strength of stud dowels, E_c: Young`s modulus of elasticity based on σ_B

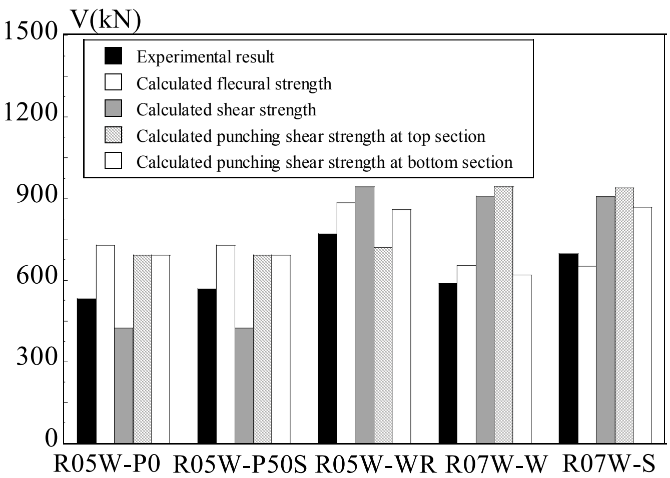


Fig. 7 Comparisons of test and calculated results

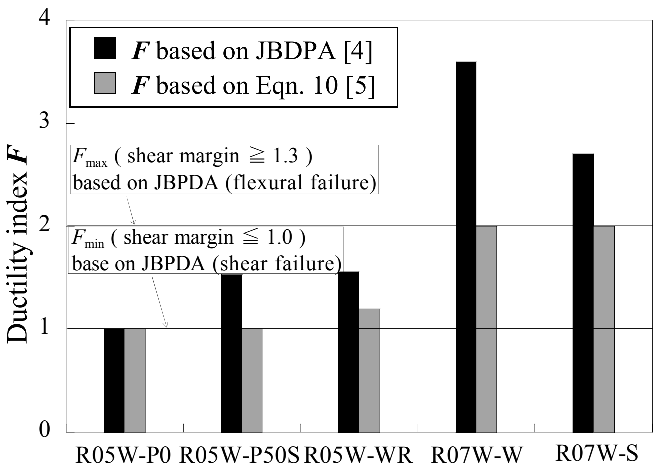


Fig. 8 Comparisons of test and calculated results

The calculated ductility indices F of the specimens based on the JBDPA design guidelines [4] and the simplified Eqn.10 [5] are presented in Fig. 8. In case of standard framed shear wall in which the shear failure is generally occurred at a drift angle, R=0.4% [6] for a shear margin (a ratio of shear forces at shear and flexural strengths) less than or equal to 1.0, the ductility index is defined as 1.0. Fig. 8 shows that the ductility index of the retrofitted specimens R07W-W and R07W-S reach to the maximum value (F=2) by JBDPA design guidelines and the values calculated by the simplified equation exceed the maximum design value.

$$F=0.6+100R \tag{10}$$

5. CONCLUSIONS

(1) In the non-retrofitted standard specimen, the typical shear failure appears in the panel wall and passes through the columns. However, by retrofitting the columns only unitizing steel plates, corner blocks and PC bar prestressing, the shear failure in column is prevented and the columns can sustain the vertical load.

(2) Through retrofit technique of thick hybrid wall by casting additional concrete sandwiched by steel plate and PC bar both the lateral capacity and ductility are improved significantly.

(3) By strengthening the top panel wall connection by casting additional concrete sandwiched by steel plates and PC bar prestressing up to beam, and by providing stud dowels at bottom, the punching shear failure can be prevented.

(4) Though stud dowels at the bottom connection can not completely prevent sliding, however, the flexural strength can be ensured and these the seismic performance can be improved.

ACKNOWLEDGMENTS

The investigation reported here in was carried out possible by the financial support of the Grant-in-aid for the Scientific Research (B), (17360272), 2005, by the Japan Society for the Promotion of Science (JSPS). The other research grants for Construction Technology, Research and Development were provided by Ministry of Land, Infrastructure and Transport, Japan in 2005 and 2006. The authors are also grateful to Dr. Md. Nafiur Rahman, postdoctoral researcher of Prof. Yamakawa's laboratory, University of the Ryukyus for his valuable assistance.

REFERENCES

[1] Yamakawa, T., Kamogawa, S., Kurashige, M. (2000). Seismic Performance and Design of RC Columns Retrofitted by PC Bar Prestressing as External Hoop, Journal of Structural and Construction Engineering, AIJ, No. 537, 107-113. (in Japanese)

[2] Yamakawa, T., Li, W. (2004). Seismic Retrofit Design of RC Columns with Spandrel Walls Utilizing Pretension High Strength Steel Bars and Steel Plates, J. of Struct. Eng., AIJ, No.577, 93-100. (in Japanese)

[3] Hotta, K., Yamakawa, T., Morishita, Y. & Rahaman, M.N. (2004). Tests and Analysis of Wing-wall Columns Retrofitted by Steel Plates and Pretension PC Bars, Proceedings of The Japan Concrete Institute (JCI), Vol. 26, No. 2, 1309-1314. (in Japanese)

[4] Japan Building Disaster Prevention Association (JBDPA). (2001). Standards and Guideline for Seismic Evaluation and Retrofit of Existing Reinforced Concrete Buildings.

[5] Otake, N., Yamamoto, T., et al. (2003). Buildings Research on New Outside Strengthening Method Contained Steel Plate Concrete Member : (Part 11), Annual Meeting of AIJ, 717-718. (in Japanese)

[6] Tomii, M., Takeuchi, M. (1968). The Relations Between the Deformation Angle and the Shearing Force Ratio (0.8 - 1.00) with regard to 200 Shear Walls, J. of Struct. Eng., AIJ, No. 153, 41-48.