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SEISMIC STRENGTHENING METHOD WITH EXTERIOR CONCRETE MEMBER INCLUDED STEEL PLATE

Yuriko HATTORI [1], Yasutoshi YAMAMOTO [2], Tadaki KOH [3]

SUMMARY

This paper focuses on a new technique based on exterior retrofit installation to middle-rise buildings. An improved method of seismic strengthening of existing R/C buildings using reinforced exterior members has been developed in the past study. Addition of exterior reinforced concrete unit is called concrete member included plate (CMIP) which is a new method of seismic rehabilitation. This method shortens the construction period and allows residents to continue living during under construction.

CMIP improves significant lateral stiffness, strength, overall ductility and energy dissipation capacity. The obvious effectiveness proves its applicability in the seismic retrofit design of R/C frame structures. Selected results are herein summarized in order to underline different parameters which influenced the behavior of the strength and/or failure mode. Experimental results are presented and discussed.

INTRODUCTION

In the aftermath of a destructive Hyogo-ken Nanbu earthquake in Japan, there is increased awareness for the need to evaluate and improve seismic performance of existing R/C buildings. Many R/C apartment buildings in 60s and 70s need seismic structural upgrade in order to match the minimum requirements of presented building codes[1]. Generally, the column with wing wall represents the critical and brittle elements of the seismic design which proved by past earthquake lessons.

However, retrofitting of wing wall becomes effective member for the strength-resistant failure mechanism. To investigate use of wing walls as a seismic retrofit measure to prevent brittle column failure, one third of actual scale model of test programs were built and tested.

The test program consists of three different specimens, which are existing typical R/C frame of one span and two stories model specimen of [B1] and seismic strengthened with CMIP specimens of [B2] and [B3]. Two cases of strengthening with CMIP are designed. The difference among the strengthened specimens relate to the level of strengthening which design to verify expected steps of the strength hierarchy. The performances of strengthened specimens are compared with the existing R/C frame specimen. The effectiveness of strengthened pattern on hysteresis response is discussed. Based on these experimental

[1] Hattori Sekkei Engineering, Inc., Saitama, Japan, yuri-hat@smile.ocn.ne.jp
[2] Dept. of Architecture, Shibaura Institute of Technology, Tokyo, Japan, yamamoty@sic.shibaura-it.ac.jp
[3] Graduate School of Science and Technology, Meiji University, Tokyo, Japan, koh@isc.meiji.ac.jp

results and analytical studies, this paper explores the beneficial effects of seismic behavior of the column with wing wall seismic retrofitted by CMIP technique.

1. TEST PROGRAM

1.1. Specimen Detail

In choosing the dimensions of the specimen, typical frame and geometrical ratios were taken in account even though some scaling was adopted to maintain the specimen size and weight to a manageable level. The specimen's cross sections and the steel reinforcement used are shown in following figures. The model of existing R/C specimen design was carried out before the recommendation of a building code of 1981 as the representative of a larger number of existing apartment buildings.

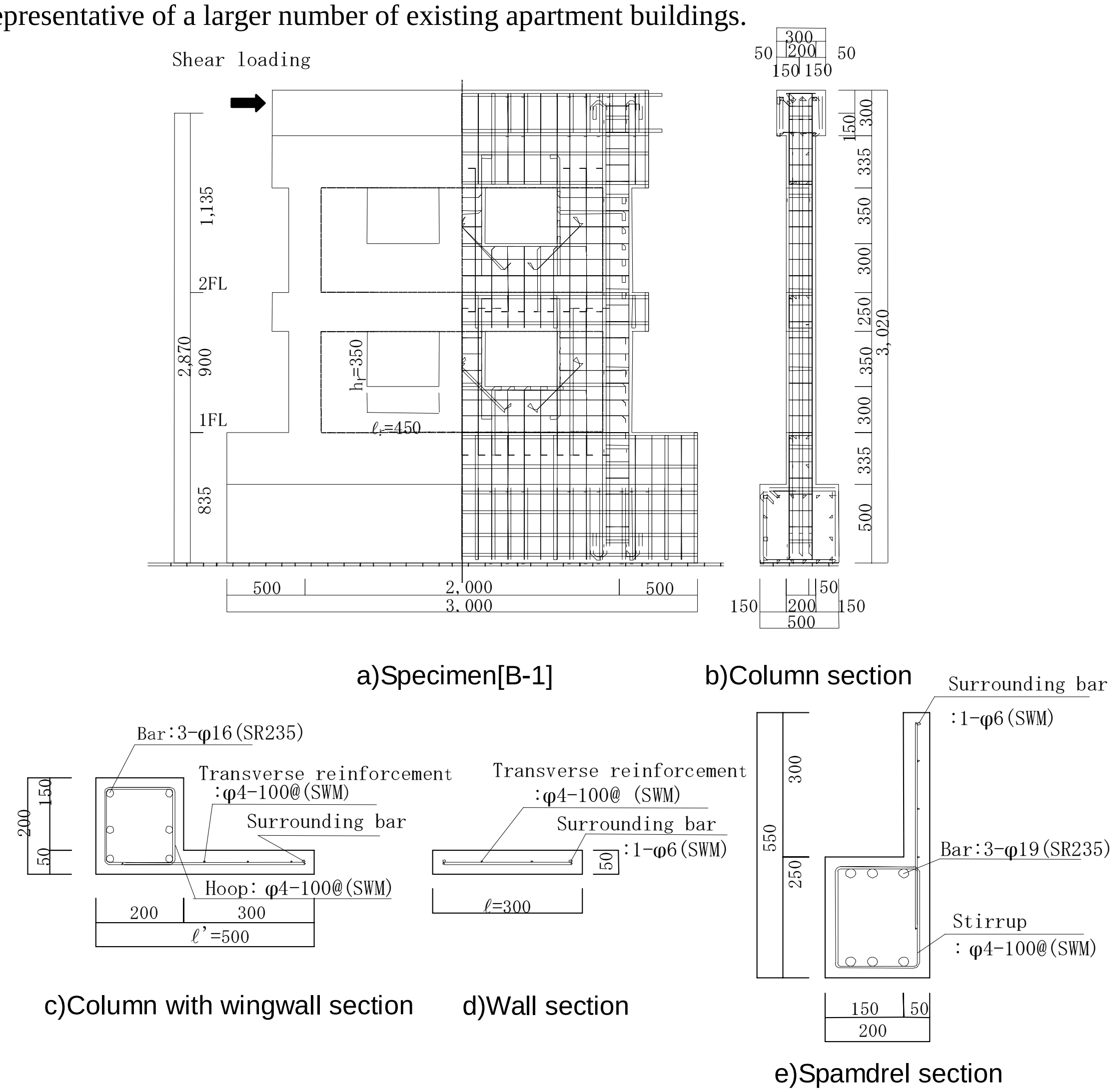


Fig.1 Existing R/C frame geometric dimensions

(1) Specimen [B-1]

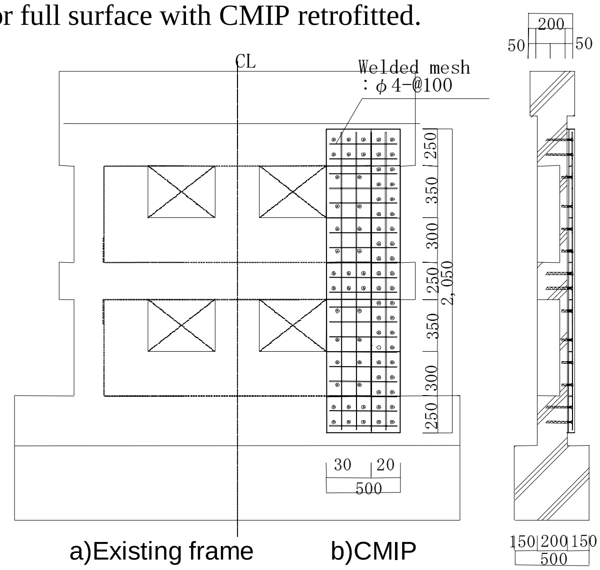
Frame model scaled to one third of actual scale of R/C frame structure with a span of 2000mm, and a story height of 900mm. column section with 200mm square with 50mm thickness wall with 300mm width, which column the longitudinal reinforcement consists of 3-16φ(p_t=0.965%), the hoop consists of 2-4φ and the equal spacing of100mm(volumetric hoop ratio p_w=0.101%),which wall the longitudinal and vertical reinforcement consists of 4φ(p_s=0.251%), which opening the surrounding bar consists of 1-6φ, the spandrel wall with 50mm thickness and 300mm height, the window opening with $h_0 \cdot \ell_0$=350×450.The shear span ratio $h_0/(2\ell')$=0.35, the center wall between openings $t_w \cdot \ell$ =50×300($h_0/(2\ell)$=0.58). The details are shown in Fig.1 c)-e).

(2) Specimen [B-2]

CMIP retrofitted specimen [B-2] is shown in Fig.2. The strengthened area by CMIP is shown in Fig.4. Seismic strengthened specimen with CMIP is described in the following ways; both column with wing wall are strengthened with added CMIP installed anchor bolts in which contains plate (PL-1.6) welded with flange (PL-6) and stiffener (PL-3.2), while injected mortar of 10mm thickness to unite both members. The trowel mortar of 40 mm thicknesses was finished. Bonded steel anchor D13 was used for joint. A headed anchor with a nut embedded in finished mortar of 40mm thickness goes through the hole into the plate. The effective embedment length was ℓ_e=35(=2.69d_a) with flat bottom owing to thin wall of 50mm thickness, ℓ_e =130(=10 d_a) for columns and beams, under cutting with 45 degree.

(3) Specimen [B-3]

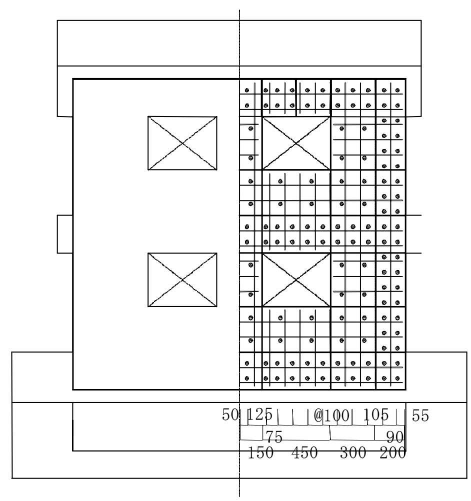
Specimen [B-3] is shown in Fig.3 and Fig.4. Installation method is the same way as specimen [B2] except for full surface with CMIP retrofitted.



a)Existing frame b)CMIP

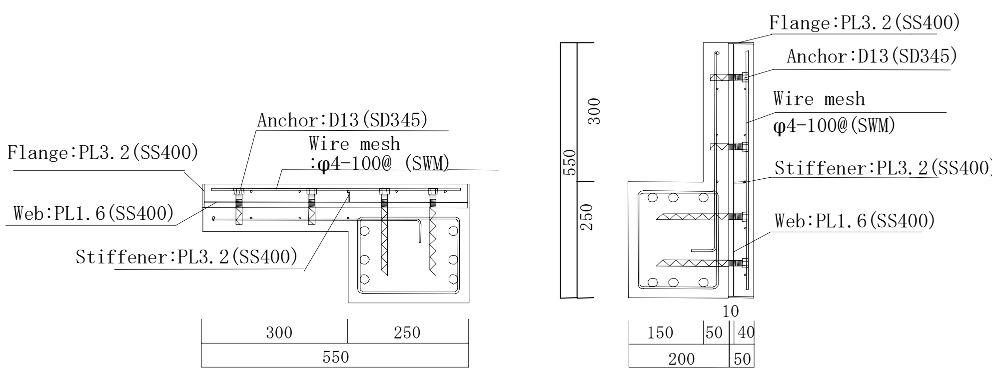
c)Column section

Fig.2 [B-2] geometric dimensions



a)Existing frame b)CMIP

Fig.3 [B-3] geometric dimensions



a)Column with CMIP section b)Spandrel section

Fig.4 CMIP retrofitted member geometric dimensions

2. MECHANICAL PROPERTIES OF MATERIALS

2.1. Concrete and Mortar

The concrete had compressive strength less than 18N/mm^2. The testing body and the base parts were cast separately. Grouting Mortar fixed embedded anchor bolts and CMIP was finished with trowel mortar. The property of concrete and mortar are shown in Table 1. Polymer cement mortar was used, it characterize low in the strength and young's modulus but rich in the ductility.

Table1 Concrete and Mortar property

Designation	Condition	Weight of unit volume	Compressive strength	Split tensile strength	Elastic modulus
		γ [kN/m^3]	σ_B [N/mm^2]	σ_{SP} [N/mm^2]	$E_s \times 10^4$ [N/mm^2]
Concrete	Existing RC	21.30	15.10	1.53	2.03
Concrete	Base part	21.60	39.20	2.87	2.34
Grout mortar	Joint	18.20	16.60	2.20	1.56
Trowel mortar	CMIP surface	18.80	22.90	2.70	1.60

2.2. Steel bars and plates

The properties of steel are shown in Table 2. The yield point σ_y was determined by using the intersection between stress-strain curve and 0.2% shifting the strain according to indefinite the value. Steel plates give low yield strength and elasticity.

Table2 Steel and plate property

Designation	Condition	Yield strength	Max. strength	Elastic modulus	Elongation
		σ_y [N/mm^2]	σ_{max} [N/mm^2]	$E_s \times 10^5$ [N/mm^2]	[%]
4 ϕ	Existing member	386.2	519.7	1.60	12.7
6 ϕ		622.5	690.6	2.03	−
16 ϕ		323.8	460.9	1.90	29.2
19 ϕ		310.0	443.2	1.85	30.3
D10		457.2	662.1	2.02	18.0
D22		383.8	589.7	1.99	14.3
4 ϕ	CMIP member	426.1	457.9	1.71	−
D13		438.2	635.2	2.18	−
PL-1.6		180.6	297.5	2.00	42.0
PL-3.2		235.8	333.9	1.78	41.0
PL-6		207.7	338.1	1.77	35.0

3. TEST PROGRAM AND OBSERVATION

3.1. Loading system and test setup

Test setup was the same for all specimens which are shown in Fig.5. A constant vertical load 118kN (σ_N=2.94N/mm^2) was applied during shear loading. The lateral load is applied by a horizontal actuator which is manually controlled, so that a variable and proportional lateral force may be applied during the test. The load cell was positioned between the hydraulic jack and the setup frame. The setup frame was restrained from rotation with two reaction poles, tied-down to the laboratory strong floor. The ductility index F value is approximately estimated by the equation of F=0.6+100R(R: story drift angle).

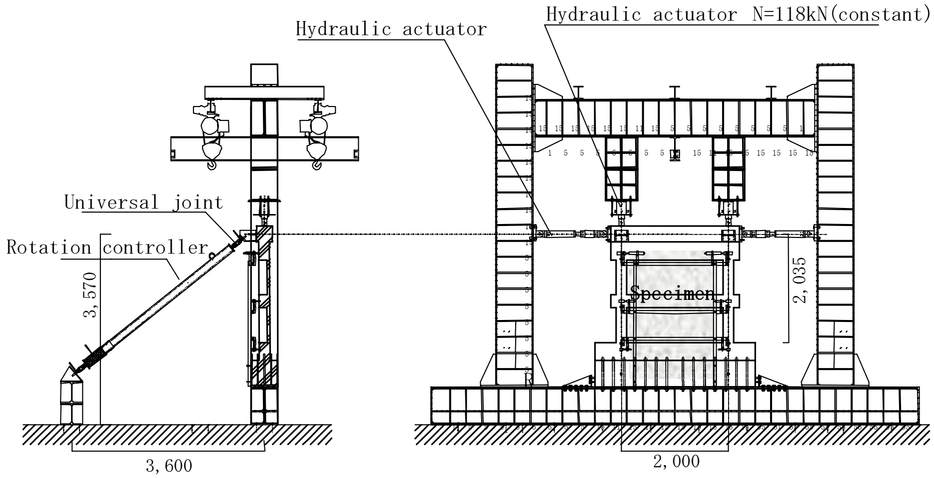


Fig.5 Test setup

3.2. Loading arrangement

The loading steps are described in Fig.6. The temporal load application recorded during the test.
Each test is characterized by constant vertical load applied to the column. The lateral load was controlled by displacement at loading steps to the story drift angle R from ±1/1000 to ±1/50 until the column failure occurrence or reached the 13th step. The flexural index (F) is indicated along with values.

Load step No.	Story drift angle R[rad]	Ductility index F
Step 1	1/1000	0.7
Step 2,3	1/500	0.8
Step 4,5	1/250	1.0
Step 6,7	1/125	1.4
Step 8,9	1/83	1.8
Step 10,11	1/63	2.2
Step 12,13	1/50	2.6

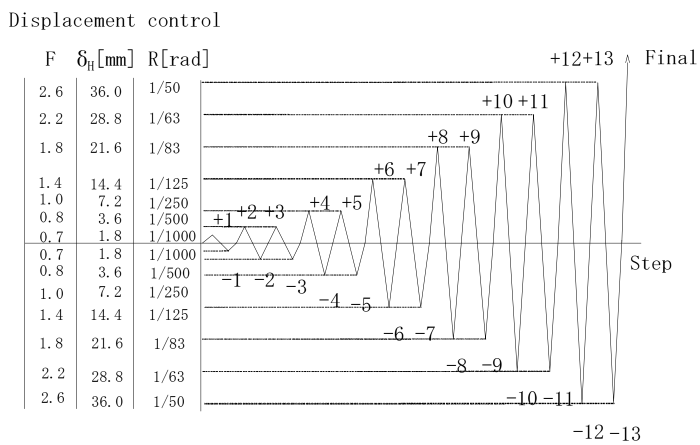
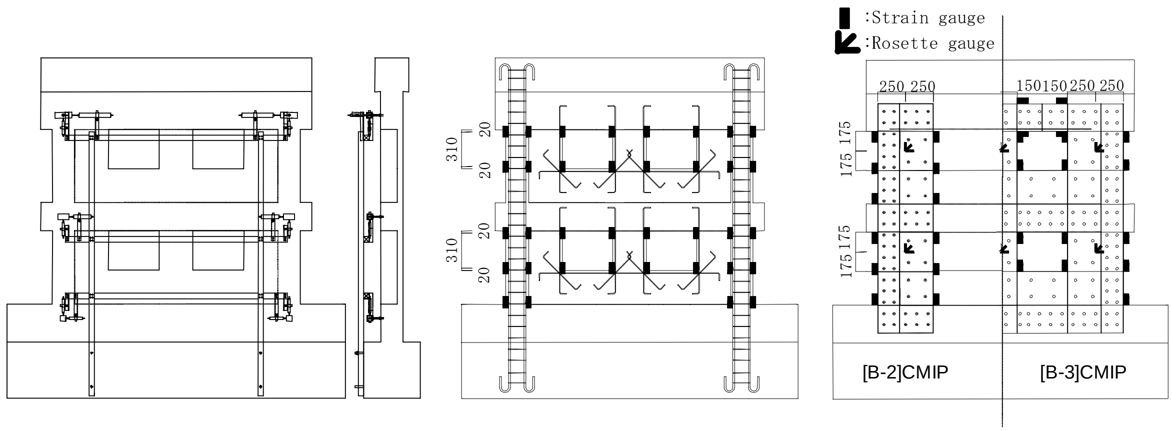


Fig.6 Loading steps

3.3. Displacement and strain measurement

The recorded forces and displacements included those measured by the load cell and linear variable displacement transducer. Fig. 7 a) shows displacement transducer placement. The devices were installed to obtain the lateral and axial displacements of the beam column joint at the top and mid-height respectively. The strain gauge for longitudinal reinforcement and rosette gauge for steel flange arrangements are shown in Fig.7 b) and Fig. 7 c) to measure flexural stress and shear stress.



a) Displacement transducer　　b) Strain gauge　　c) Rosette gauge

Fig.7 Displacement transducer and gauge arrangement

4. TEST RESULTS

4.1. Load-displacement relationship

The figure shows the horizontal force-displacement hysteresis curve at the loading level. Different stages for each specimen are shown in the Fig.7.

(1)Specimen [B1]

The diagonal cracks occurred in the wing wall and center wall at the beginning of loading, the flexural cracks in walls occurred before reaching the maximum strength. The maximum shear strength was (+230.6kN, 7.20mm) and (-225.8kN, 7.30mm).

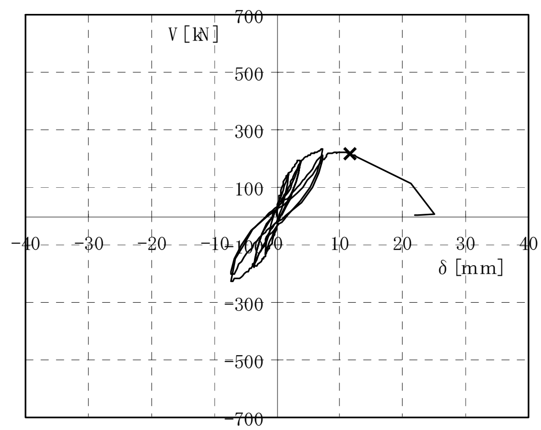
The F value index at fracture point was 1.03 (+206.7kN, 12.65mm, R=1/133).

The applying load came to uncontrolled and the specimen failed in shear at compression-side column.

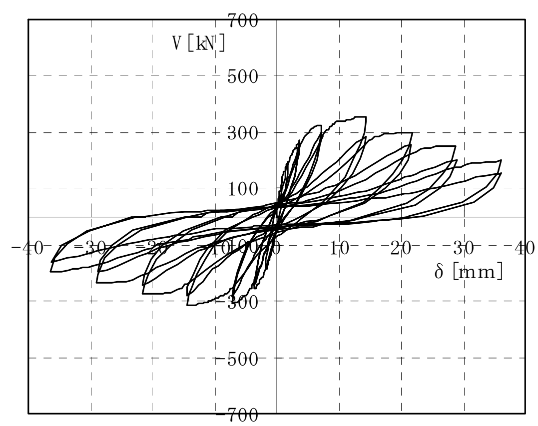
(2) Specimen [B2]

The crack at initial step occurred in the same region as [B1] excluding cracks in CMIP panel area occurred at 72-78% of the maximum shear load. The hysteresis curve became S-shaped at the 5th step, R=1/250, 92-99% of maximum load. The crossed shaped diagonal crack in the middle narrow wall occurred at the same deformation of R=1/250(F=1.0) without degrading of shear capacity. The deterioration of strength gradually increased after the 7th step with R=1/125.

Cracks expanded and bonded failure occurred in the spandrel surface remarkably at the 8-9th steps with R=1/83. The upper side of spandrel wall failed in shear compression conspicuously. It resulted in deterioration of the strength brought by increase in



a)[B-1] V- δ curve



b)[B-2] V- δ curve

deformation. CMIP cracked slightly at nearly flexural tension of the 10-11th steps with R=1/63.

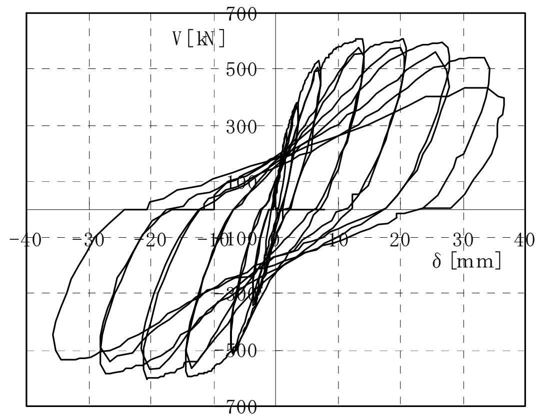
(3) Specimen [B3]

The diagonal cracks spread in the middle wall area which was lager than that of [B2] damaged process.

The hysteresis curve indicated a linear slope until R=1/1000. Cracks increased damage with shear loading. After reached at the maximum strength of 98.5-100% the deterioration of strength and the hysteresis curve are declined.

Maximum strength and energy absorption capacity were remarkably improved.

The graphs reported in Figures show all this.



c)[B-3] V- δ curve

Fig.8 Load - displacement curves

4.2. Story drift angle and horizontal torsion

4.2.1. Story drift angle

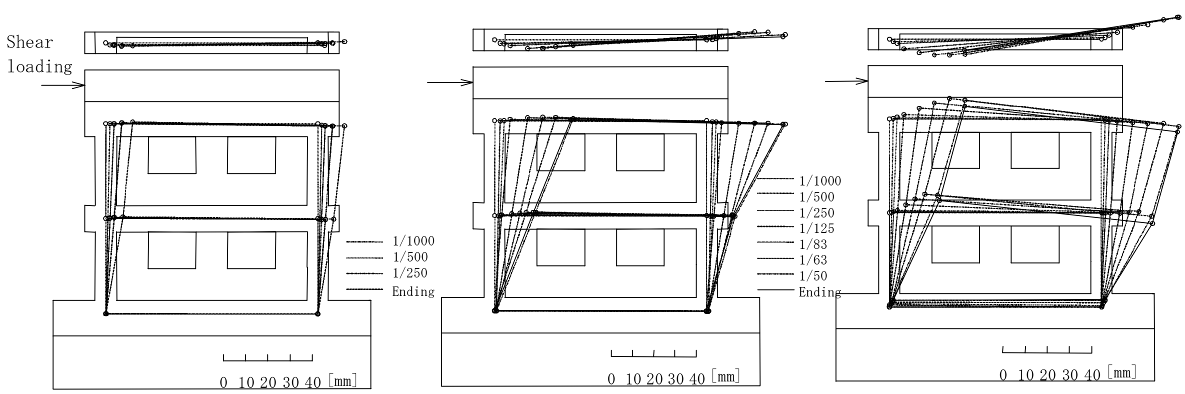
Fig.9 illustrates the behavior of story drift angle and horizontal torsion at loading steps of test units.

[B-1] presents the similar amount of deformation as the shear displacement. [B-2] presents larger deformation in 2nd story due to the effect of flexural deformation of the wing wall. [B-3] presents the effect of flexural behavior after shear compression failure of the existing wing wall at the 11th loading step, consequently the slip deformation at 1st story surpassed 2nd story.

4.2.2. Top of the column torsion

Fig.9 describes the behavior of column's top torsion.

It is considered that [B-1] behaved to avoid the torsion by no eccentric moment from CMIP reinforcement. The eccentricity became obviously clear according with the amount of CMIP retrofitted member, however it has not involved in the failure mode.



a) [B-1] b) [B-2] c) [B-3]

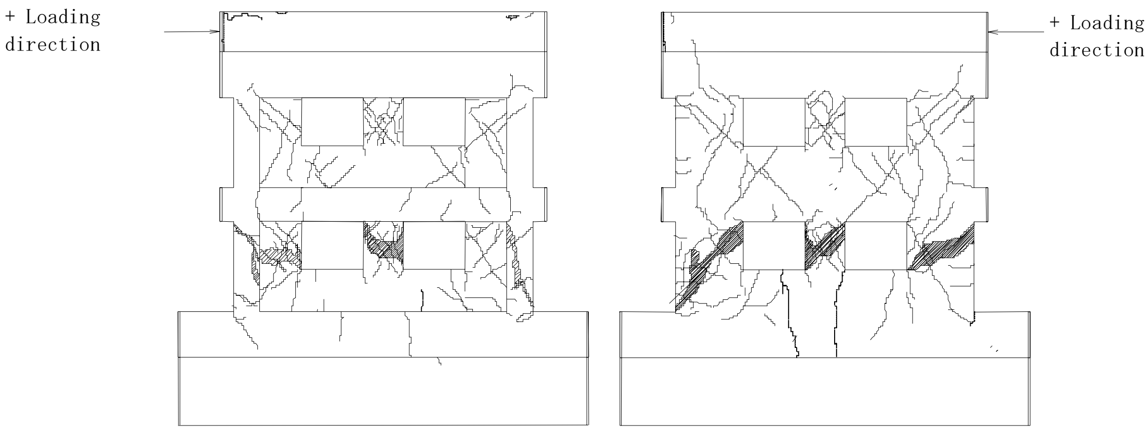
Fig.9 Story drift angle and top of the column torsion

4.3. Failure process

Figures illustrate final crack patterns and failure mode.

(1) Specimen [B-1]

Slight shear cracks have observed at R=1/500(F=0.8) without the deterioration of the strength. Increasing the crack with R value increases, the lack of confinements and shear slippage were progressed.

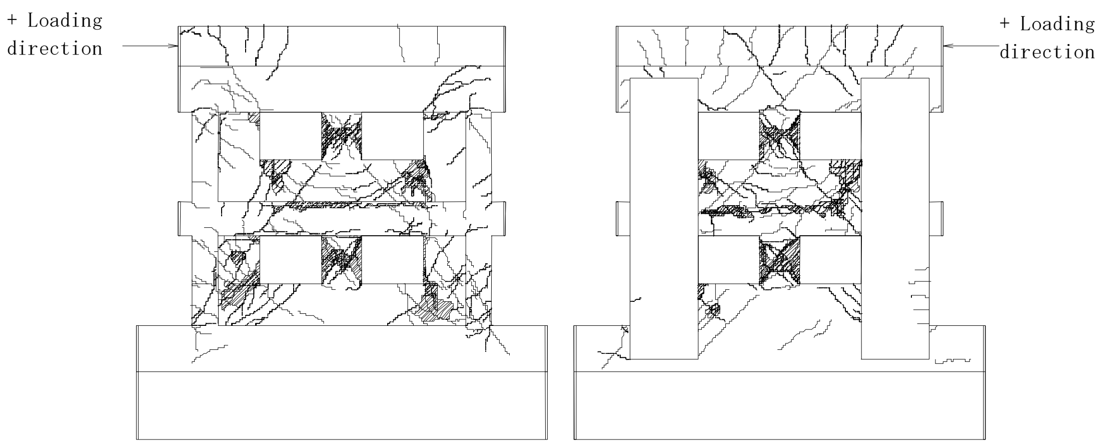


a)Existing member side b)CMIP retrofitted side

Fig.10 [B-1] **Final crack pattern**

(2) Specimen [B-2]

The crossed shaped diagonal crack occurred in both front and back side at R=1/500(F=0.8). The cracks increased with approaching the maximum strength. CMIP reinforcement turned to behave as flexural deformation. After the 5th step at R=1/125(F=1.4) was the maximum strength recorded, conspicuously cracks increased in 1st story wing wall. The center wall started spalling and lost the shear capacity. Clearly the bottom wing wall at 1st story was destroyed with R=1/83(F=1.8). Subsequently, bond failure occurred in 2nd story of spandrel. Residual deformation within R=1/200 will possible to retrofit, however R=1/50(F=2.6), residual deformation over R=1/100 did remarkably decreased the strength. Therefore F value index of [B-2] was definite R=1/83(F=1.8).

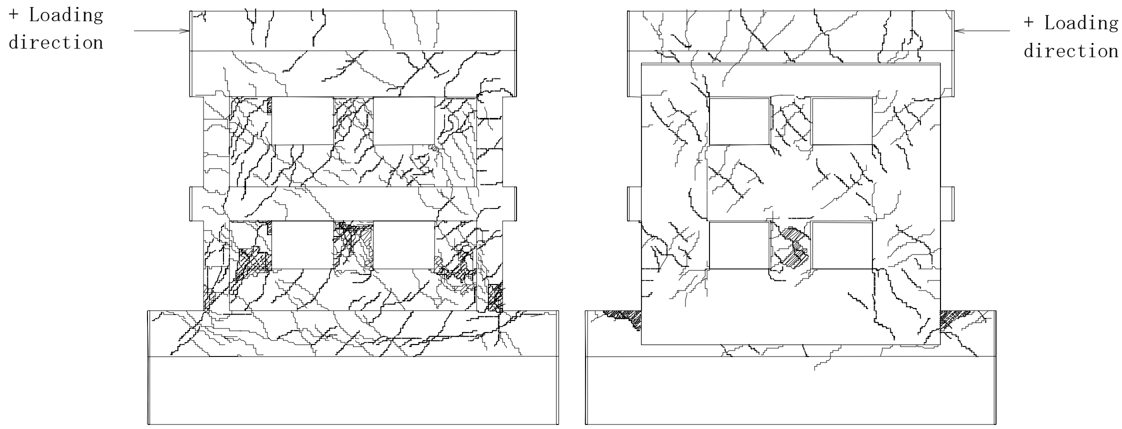


a)Existing member side b)CMIP retrofitted side

Fig.11 [B-2] Final crack pattern

(3) Specimen [B-3]

Flexural and shear cracks were observed at the 1st story wing wall in tension with R=1/500(F=0.8). Expanding cracks with R value at 1/250, first cracks occurred in CMIP member and other cracks went through the stories. The frame behaved flexural yielding mode at R=1/125(F=1.4) and cracks increased. The cracks spread out at R=1/63(F=2.2), and the shear failure of wing wall under compression was showed. The column and wing wall behaved decisively shear failure at R=1/50(F=2.6). In the strengthened specimen of [B-2] and [B-3], the steel panel failure was observed before reaching the ultimate shear capacity, the ultimate column shear strength was 1.53-2.63 times of the virgin column. These results can be clearly observed in Figures.



a)Existing member side b)CMIP retrofitted side

Fig.12 [B-3] Final crack pattern

4.4. Maximum shear strength and failure mode

Table 3 presents the failure mode at the maximum shear load. However, after the ultimate state level, the response of specimen [B1] went significantly uncontrollable due to the lack of axial resistance, while specimen [B2] and [B3] were kept adequate strength until the fracture. Shear strength of CMIP contributed the shear capacity, ductility, and energy absorption to be improved.

Table3 Maximum load and failure mode

Specimen	Loading direction	Ultimate strength	Displacement at V_u	Story drift angle at V_u	Ultimate state level strength	Displacement at V_L	Story drift angle at V_L	Failure mode
		V_u [kN]	δ_u [mm]	R_u [rad]	V_L [kN]	δ_L [mm]	R_L [rad]	
B-1	+	230.0	7.20	1/250	206.7	12.65	1/142	Shear slip at 1st story frame
	−	225.8	7.30	1/247	-	-	-	
B-2	+	352.6	12.48	1/144	298.9	21.72	1/83	Bonded failure at 2nd story spandrel
	−	313.8	14.32	1/126	275.1	21.53	1/83	Bonded failure at 2nd story spandrel
B-3	+	606.3	14.41	1/125	543.1	34.31	1/52	Shear compression failure at 1st story bottom column
	−	605.6	21.65	1/83	543.8	35.38	1/51	Shear compression failure at 1st story bottom column

5. DISCUSSION ON TEST RESULT

5.1. Analysis of shear stress distribution for existing frame and CMIP

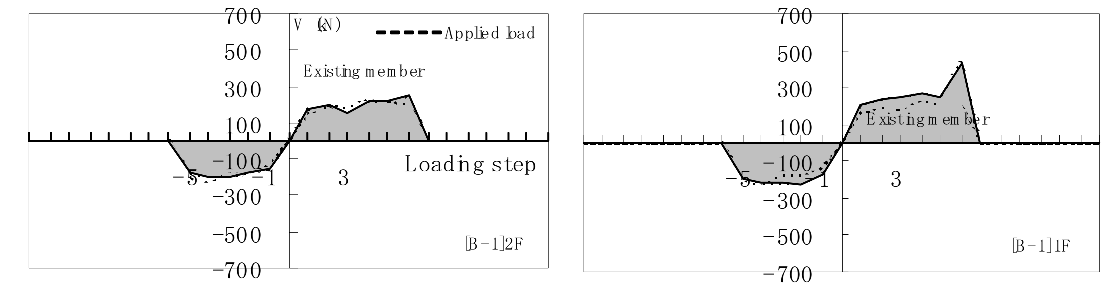
5.1.1. Moment curvature analysis

(1)Existing R/C member

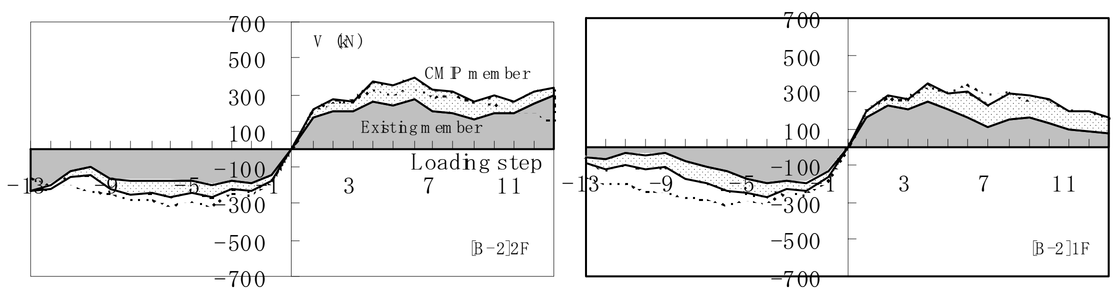
Reinforced concrete analysis for axial force and bending moment is performed at the top and bottom column section to idealize the stress-strain behavior of the concrete with a rectangular stress block to simplify the calculations. The stress distribution is assumed a rectangular stress block within a distance equal to the neutral axis and a resistant force equal to the fraction of the concrete compressive strength. A unique bending moment can be calculated at this section curvature from the stress distribution. From the analysis method which calculated shear strength performance of existing R/C member is shown in Fig.13.

(2) CMIP reinforcement

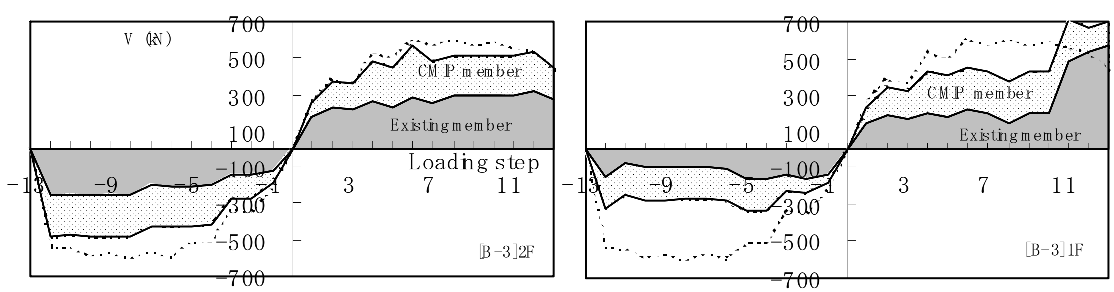
The Shear stress of CMIP was calculated from Mohr's circle model using strain data obtained by rosette gauge. The shear strength is accurately determined by the area of steel plate led by Tresca stress criteria.



a)Specimen [B-1]



b)Specimen [B-2]



c)Specimen [B-3]

Fig.13 Comparison between theoretical value and tested shear resistance

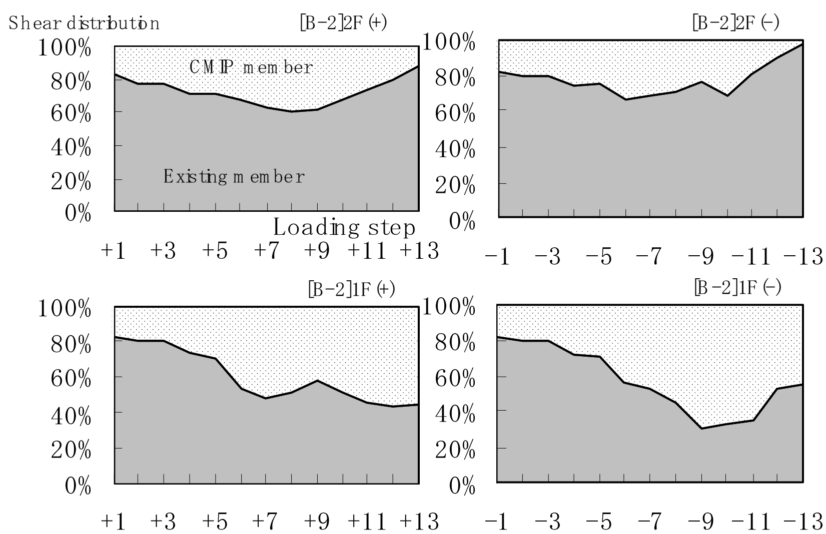
5.1.2. Determination of shear strength

(1)Shear resistance calculation

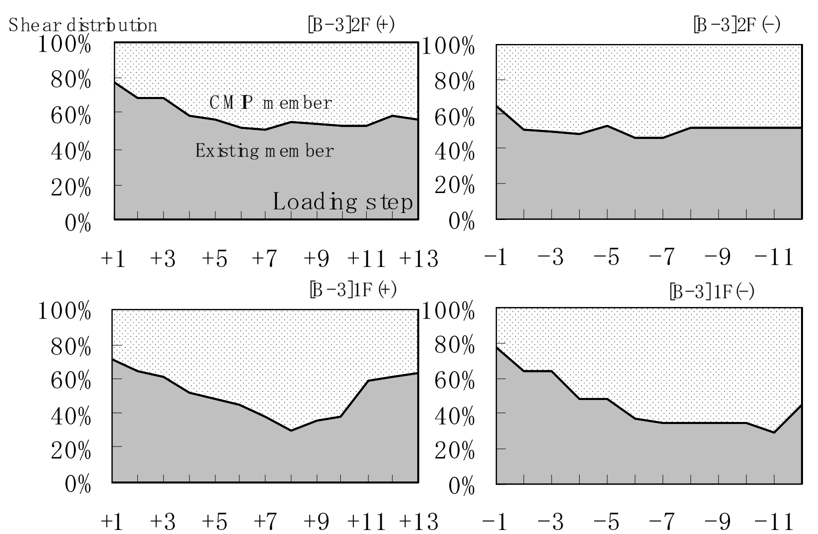
Fig.13 indicates shear resistance determined by moment curvature analysis which distributed by existing R/C frame member and CMIP compared with the recorded shear load. The shear load behavior roughly corresponding with the test results are verified. Approaching the final loading step, shear resistant in the existing R/C frame member was predominated in share. It means the influence of unapt application for Navier’s hypothesis.

(2)Shear resistance distribution

The ratio of shear strength distribution of the existing frame member and CMIP are shown in Fig.14. The shear load increased with the amount of dominant distribution ratio of CMIP. Particularly it is clearly observed at 1st story.



a)[B-2]



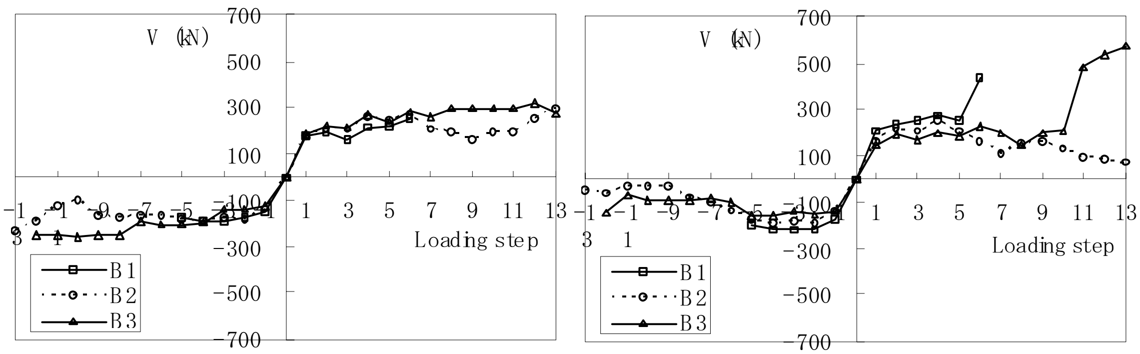
b)[B-3]

Fig.14 Shear force distribution value

(3)Evaluation of shear resistance of existing R/C frame

Fig.15 illustrates the value of shear resistant share of existing R/C frame member compared with virgin specimen and strengthened specimens. These specimens bear the similar shear capacity independent on the amount of strengthened area. The deterioration of the shear strength for [B-2] and [B-3] have not observed after surpassing the fracture value of [B-1]. Strengthened specimens maintain the shear strength after the cracks occurred in the wing wall. As the result steel anchors contribute the unity of existing and strengthened member.

The shear strength distribution of CMIP member increases with the deformation. Existing R/C frame member of [B-2] and [B-3] maintain the shear strength capacity and improve the ductility.



a) 1st story a) 2nd story

Fig.15 Shear strength contribution of existing R/C frame

5.2. Comparison between theoretical prediction and test result

Test results confirm that the test setup works as expected and the behavior of tested specimens was consistent with the theoretical prediction. As experimentally observation, the analysis confirms that the failure mode is designed to be determined by shear slip deformation and yield of beam, unless the fracture of CMIP member. The theoretical values calculated by the diagnosis standard code in Japan[1] and safety factors are compared and shown in Table 4.

Table 4. Comparison between theoretical prediction and test value

Specimen	Value	Failure mode	Ultimate strength V_u [kN]	V_u/Theory value Vu/V_D [kN]	Ductility index F^{*1}
B-1	test	Shear slip	230.0	−	1.00
	theory	Ultimate shear strength*2	268.1	0.86	1.00
		Slip strength*3	223.6	1.03	0.80
B-2	test	Beam shear failure	352.6	−	1.80
	theory	Beam shear failure	381.7	0.92	1.50
		Shear break out strength of anchor bolt*4	328.6	1.07	−
B-3	test	Flexural failure	606.3	−	2.52
	theory	Panel shear failure*5	599.3	1.01	−
		Yielding strength of anchor bolt*6	599.5	1.01	−

*1 Estimated value is given by F=0.6+100R

*2 Ultimate shear strength(V_{SU}) is given by $\Sigma V_{SU} = {}_{column}V_{SU} + {}_{column}V_{MU} + {}_{wall}V_{SU}$

*3 Slip strength of punching shear column(V_{slip}) is given by $\Sigma V_{slip} = \kappa \cdot \tau \cdot b \cdot D$ κ: Ratio of average shear span

*4 Break out anchor strength is given by top and bottom of panel anchor's resisting moment

*5 Panel shear strength is given by plate formulas

*6 Yielding strength of anchor is given by bottom of panel anchor’s resisting moment

The CMIP retrofit system was designed to yield beams in flexure prior to the column remain the shear capacity. The system showed ductile response during the test so that it assumes the column and wing wall have not fractured.
The theoretical shear strength of specimen [B1] was 268.1kN and the test result was 230.0kN, the gap was modified by applying theory of shear friction. Specimen [B2] and [B3] have obtained the closer values according with the failure mode by theoretical formula.

6. CONCLUSION

Results of the test program confirmed that the CMIP method promises the elastic seismic strengthening technique for wing wall in middle rise apartment buildings. In fact, CMIP member improves shear strength and ductility which proved by experimental study
An important element in the design of CMIP method is the integration of the new materials with the existing materials fastened by bonded anchors. This approach has succeeded in delaying the column shear failure mechanism and shifting the beam yield failure mechanism by reduction of story drift levels.
The study shows that CMIP scheme provides effective seismic resistance and identifies seismic retrofitting strategy and simplified design methodology as important areas for continued research.
The goal of the project is to define a design criterion that is shear transfer mechanism through bonded steel anchors. So that, known the initial conditions and loads of CMIP and the joint anchors, it will possible to design the CMIP structural seismic upgrade which contributes the failure mode and improves the strength and ductility.
The experimental research of this program continued to be developed its applicability in the seismic design of R/C frame structures by experimental study.

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