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EXPERIMENTAL STUDY ON SEISMIC RETROFIT OF R/C BUILDINGS USING EXTERNAL STEEL PORTAL FRAMES

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SUMMARY

A large number of old R/C buildings to be reinforced against severe earthquake exist in Japan. In this paper, authors present a new reinforcing method by using steel portal frames (Portal-Grid method). This study started by focusing that the modulus of elasticity of steel is about 10 times of concrete. By utilizing this fact, steel portal frame with smaller section compared with R/C section can add sufficient rigidity and strength to weak R/C frame without braces. The portal frames are composed of build-up H-sections, whose rigidity are adjusted to be equal to the rigidity of R/C sections. Steel portal frames are connected to outer surface of R/C building. By the experimental approach, authors proved that the strength of R/C frames is highly improved by using Portal-Grid method. And the strength and rigidity of the reinforced frames can be calculated to sum up strength and rigidity of portal frames and of R/C frames.

INTRODUCTION

In Japan, the lack of the seismic adequacy of older R/C buildings, which are inadequate for the modern seismic design codes, is a matter of growing concern. These R/C buildings were seriously damaged in the Hyogoken-Nanbu Earthquake (January 1995). After the enforcement of the Seismic Retrofit Promotion Law (December 1995), seismic rehabilitation has been gradually enhanced, however it still makes little progress.

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Many seismic reinforcing methods that have been developed so far are mainly structural reinforcing methods that focus on seismic upgrade. Among them, common approaches use steel bracing members for retrofitting devices. In the early stage of these steel bracing methods, H-section steel bracing members were installed inplane. However, the application of the large H-section steel bracing members or buckling preventing members caused aesthetic disadvantage for the appearance or the view from the inside. In recent years, the development of the external retrofitting method combined with the use of Force Limiting Devise (FLD) as bracing members enables to greatly shorten the period of construction and improve the structural adequacy in spite of slender members, which are still disadvantages to use buildings.

In this paper, the experiment of External Portal Steel Retrofit Method (Portal Grid Method, hereinafter referred to as PG Method) is presented. PG method overcomes these disadvantages by using external steel portal frames and by disusing bracing members (See Fig.1). The steel portal frame consists of columns and beams both are made of build-up H-section steel member. The flexural rigidity of the each column is adjusted to approximately equal to that of the as-built R/C column. Both the columns and beams are attached to the exterior of the existing R/C structure for the purpose of reinforcement with sufficient strength. Focusing attention the fact that the modulus ratio of steel to concrete is approximately 10 created the idea that built-up H-section steel of smaller section in size than the existing R/C section enables to provide sufficient rigidity and strength. Although the steel portal frame utilized in the traditional external retrofit method, was considered just as sub member to install bracing members, its rigidity and strength are counted for retrofitting in the PG Method. The steel portal frame is connected to the existing R/C structure by means of the anchor fastened to the existing frame, the stud bolts welded to the steel portal frame, spiral hoop, and non-shrinking mortar (See Fig.2).

In this paper, tests on single-story single-span R/C frame specimen under cyclic lateral loading are reported which confirm the effectiveness of retrofitting of the PG Method. The research intended to verify that the load-deformation relation of the specimen after reinforcement is calculated by simply adding the theoretical value of the R/C frame and the steel portal frame.

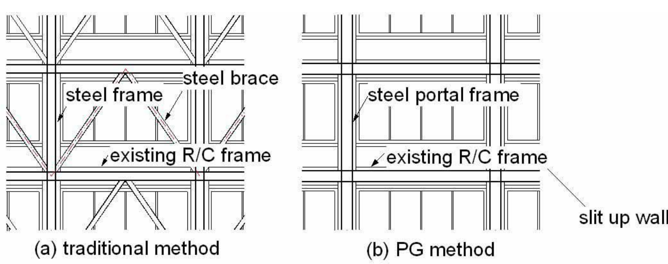


Fig.1. PG method image

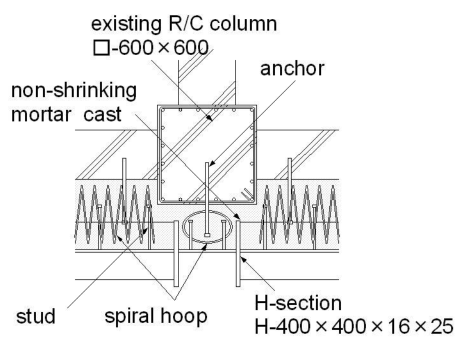


Fig.2. column section example

EXPERIMANTAL SCHEME

Specimen

Four one-third scale cut-off R/C frame specimens were constructed. Each specimen is composed of two adjoining flexure-dominant columns, footing beam, and second-story beam corresponding to first floor of five-six story R/C buildings that has floor height of 3m, and span of 5-6m. One of them was tested without any retrofit and the others were reinforced by connecting steel portal frame as shown in Fig. 3 (a) and (b), respectively.

Based on the assumption that lateral movement of the frame are restrained by the floor slab, the specimens were restrained only in out-of-plane direction by means of jig attached to the upper beam at the concrete joint panel zone on the opposite side of the steel portal frame.

The portal steel frame is connected to the R/C frame by means of studs, anchors, spiral-hoop, and non-shrinking mortar. It was designed that lateral load applied to R/C frame should be transmitted to steel portal frame at upper beam, in the same manner as in the case of retrofitting using bracing members.

The experimental parameter is the flexural rigidity of the H section columns of portal steel frame, the columns of specimen No.2 were designed to possess flexural rigidity approximately equal to that of R/C frame after early-age cracking, which is considered 70 % of the initial stiffness. With regard to specimen No.3 and No.4, approximately 75 % and 50 % of R/C frame flexural rigidity correspond, respectively.

As shown in Fig. 3, the bottom of the portal steel frame is not connected to the base. Fig. 4 and 5 shows the column and beam cross section of specimen No.2-4, anchor and stud, respectively. Joint panel zone has the same thickness as column and beam web.

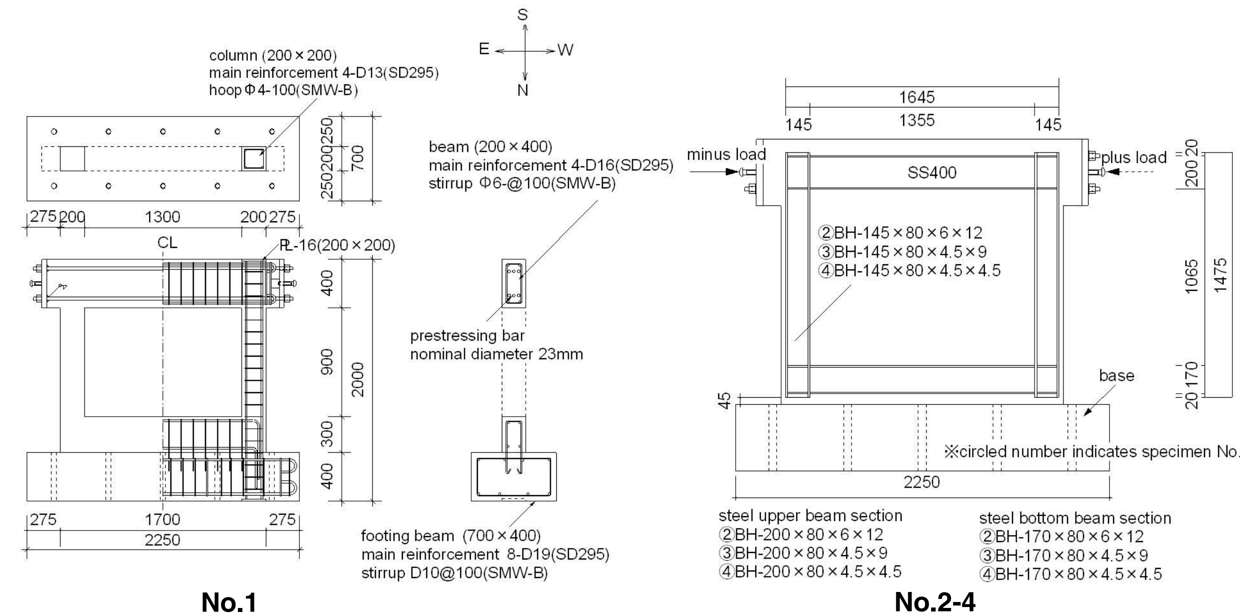


Fig.3. specimen

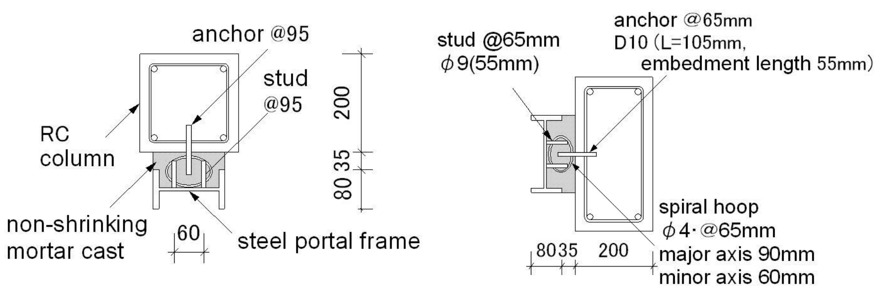


Fig.4. column and beam section

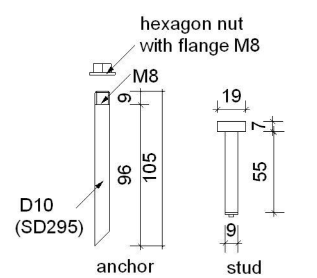


Fig.5. anchor and stud

Table 1. Build-up H sections of column

specimen No.	H (mm)	b (mm)	t_f (mm)	t_w (mm)	Is (cm^4)	EI(kg・cm^2) ※E=205(kN/mm^2)
2	145	80	12	6	940	1.97 $\times 10^9$
3	145	80	9	4.5	744	1.56 $\times 10^9$
4	145	80	4.5	4.5	450	0.94 $\times 10^9$

Assuming that the modulus of elasticity of concrete E_c is 21 kN/mm^2, the initial flexural rigidity of as-built R/C column is given by

$$E_c \cdot I_{rc} = 2.79 \times 10^9 (kN \cdot mm^2)$$

Where, I_{rc} represents cross-sectional moment of inertia of the column.
Taking the effect of early-age cracking, flexural rigidity of the column under loading is set to

$$E \cdot I = E_c \cdot I_{rc} \times 0.7 = 1.96 \times 10^9 (kN \cdot mm^2)$$

Based on this value, portal steel frames were designed. H-sections size are summarized in Table 1.

Anchors and studs designed according to reference [7] were arranged in beam half as many again as the amount to transmit ultimate story shear of specimen No.2, and in column to transmit beam shear at the same point. Anchor has effective embedment length of 55mm, and fastened to beam and column at 65 mm and 95mm intervals respectively. Stud with a diameter of 9mm, and length of 55mm was specially designed according to JIS B1198 and welded to H section at the same intervals as anchor. Spiral hoop with a diameter of 4mm was specially shaped to an ellipse with a major axis of 90mm and a minor axis of 60mm by pressing circular spiral hoop with a diameter of 75mm.

Loading scheme

Specimens were tested under constant axial load and reversed cyclic lateral load using the pantograph system setup shown in Fig. 6. As shown in Fig. 7, lateral load was applied to specimens by a 500kN oil jack via L-shape lateral loading frame and a high-tension bolt that was attached to steel plate bolted to upper beam.

Assuming these specimens represent first story of RC building, a constant axial load per a column of 168kN, which was determined to be 20% of mix proportioning strength of the concrete (21kN/mm^2), was applied through loading beam using a 2000kN oil jack. Lateral load was applied according to the loading program shown in Fig.8 under an axial load maintained constant.

Rotation angle was calculated by dividing relative lateral displacement of the adjoining beams by a distance from the crest of bottom beam to the midpoint of upper beam (1100mm). The relative lateral displacement was measured by displacement gauge 1 placed on the upper beam.

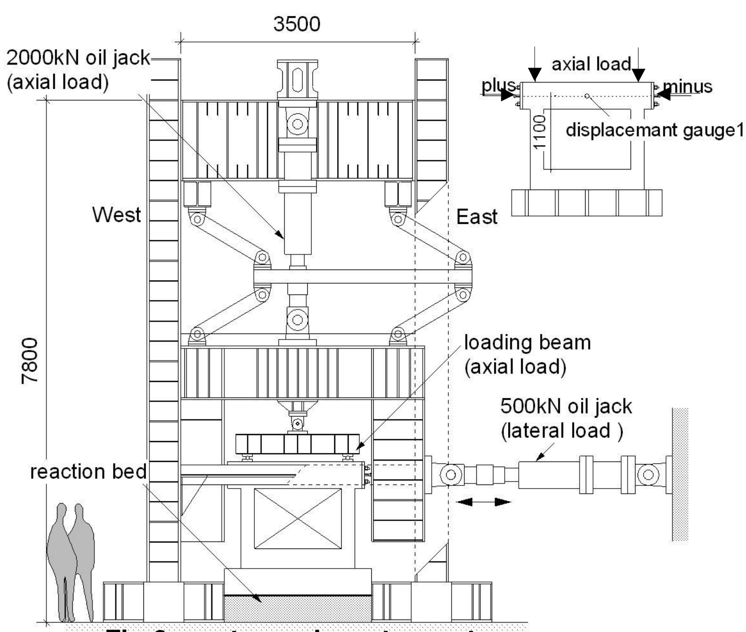


Fig.6. pantograph system setup

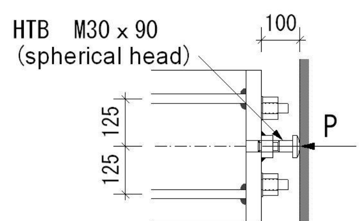


Fig.7. upper beam end

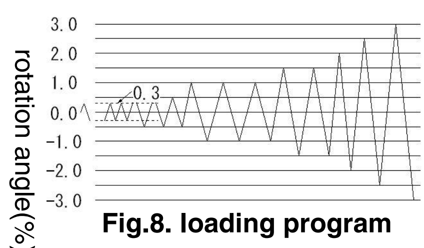


Fig.8. loading program

Displacement gauge and strain gauge

Displacement gauges and strain gauges were attached to specimen as shown in Fig.9 and Fig.10 respectively. Strain gauges were attached on east side of column and main reinforcement of RC column at the point 20mm above the crown of bottom beam.

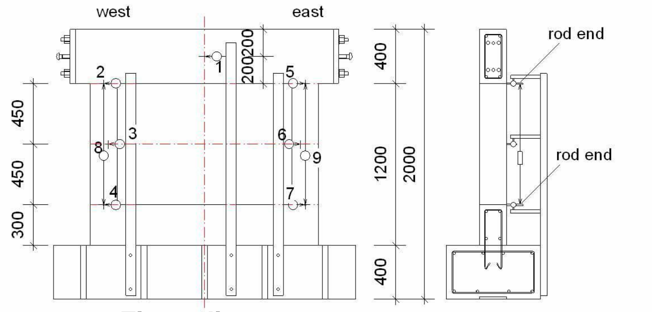
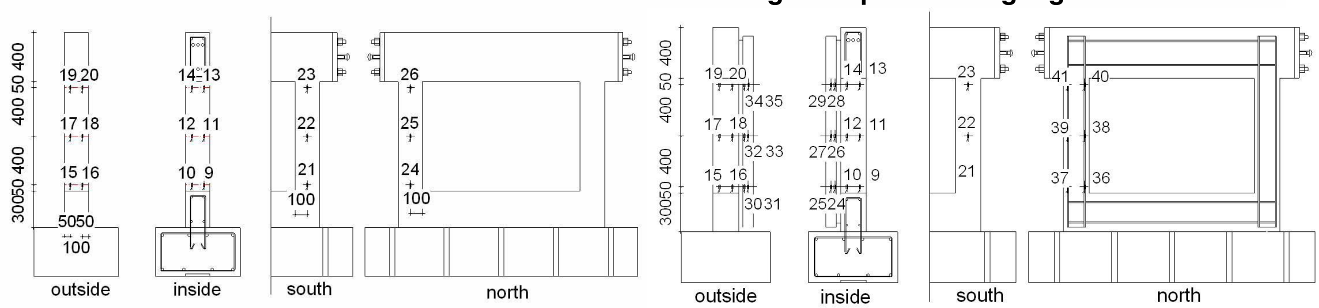


Fig.9. displacement gauge



(a) No.1 **(b) No.2-4**

Fig.10. strain gauge

RESULTS AND DISCUSSION

Material test results

Material test results are summarized in Table 2.

Table 2. material test results

concrete

material age	compressive strength (N/mm^2)	modulus of elasticity (kN/mm^2)
46th	25.7	22.9
50th	26.2	22.5
53rd	26.8	22.5
average	26.2	22.7

mortar

material age	compressive strength (N/mm2)	modulus of elasticity (kN/mm2)
25th	63.7	23.3
29th	70.4	—
32nd	62.6	24.1
average	65.6	23.7

steel

	yield stress (N/mm^2)	maximum stress (N/mm^2)	modulus of elasticity (kN/mm^2)	fracture elongation (%)
main reinforcement	350	487	187	—
hoop	688	692	247	—
steel (t=12)	295	433	206	42.1
steel (t=9)	289	419	206	42.7
steel (t=6)	248	334	206	49.9
steel (t=4.5)	291	367	210	44.6
stud	366	460	272	—
anchor	330	457	180	26
spiral hoop	513	548	190	—

Concrete and mortar material tests were conducted at the age of day 46th, 50th, 53rd and at the age of day 25th, 29th, 32nd respectively, which were in conjunction with the experimental day. There were little differences in the compressive strength and the modulus of rigidity of concrete and mortar in each age, and therefore, the average of each age was adopted in the following discussion.

Steel column stress at the point of axial load application

Unlike the case of field application where steel portal frame is attached to existing RC structure, some axial load was applied to steel portal frame because it was attached to RC frame prior to the application of axial load in this experiment. Therefore, the contribution of axial load to steel column must be confirmed in the experiment. The stress of steel column flange at the point of initial axial load application was calculated from the average strain of steel column flange measured with strain gauge attached on it. The contribution of axial load to steel column was evaluated as the ratio of the stress in initial axial loading condition to the yield stress of flange. As shown in Table3, the contributions of axial load in No.2 and No.3,4 was approximately 1% and 3% respectively and tend to increase a little from top to bottom of steel column in all specimens. They are taken into account to estimate full plastic moment as described later.

No cracking and gap between steel portal frame and RC frame were observed during the application of axial load.

Table 3. stress contribution of steel column when axial load applied

	bottom		middle		top		yield stress of flange (N/mm^2)
specimen	stress when axial load applied (N/mm^2)	stress ratio	stress when axial load applied (N/mm^2)	stress ratio	stress when axial load applied (N/mm^2)	stress ratio	
No.1	—	—	—	—	—	—	—
No.2	3.96	1.34	3.12	1.06	2.19	0.74	294.7
No.3	9.34	3.23	9.27	3.21	8.83	3.06	288.8
No.4	9.92	3.41	9.76	3.35	7.47	2.57	291.0

※ stress ratio=stress when axial load applied /yield stress of flange

Failure mode and load-deformation relationship

Failure process and load-deformation relation are shown in Table 4 and Fig.11, respectively. The damage of specimens such as significant shear failure at the base of column of specimen No.1 and X-shaped crack occurred with cyclic loading of specimen No.2-4 are illustrated in Photo 1 (a) and (b) respectively.

Apparently, the gap between the center of loading setup and the center of rigidity of specimen had no influence with displacement or failure because of adequate restrain by the jig shown in Fig. 6.

Fig.12 shows the crack of specimen No.1 and No.2 at rotation angles of 1.0% and 2.0%. The maximum lateral load of specimen No.2-4 that were reinforced with steel portal frame increased by a factor of 2-2.5 compared to No.1, as built specimen. The point of shear failure of RC column at the base (●) and final cycle are shown in Fig.11. No.1 maintained a constant lateral load until the cycle of ±2.0% rotation angle after buckling of RC column main reinforcement, which is flexural failure mode. At the final cycle (2.5% rotation angle), shear failure occurred at the base, which caused significant drop in load resistance. No.2-4 reached each maximum strength at the cycle of ±2.0% rotation angle, which was the last deformation level at which No.1 could maintain a constant lateral load, and subsequently the lateral load resistance started to drop gradually as the flexure cracks became wider. They kept stable behavior under cyclic loading without significant drop of the lateral load resistance even at the final cycle (3.0% rotation angle). Specimen No.2-4 maintain the lateral load resistance with significant increase of it and rigidity after RC column became unsupportable, which indicates that the stress applied to RC frame shifted to portal steel frame.

Table 4. failure process

specimen	RC frexural cracking load		main reinforcement yield load*		steel column yield load*		maximum lateral load	
			column inside					
	rotation angle (%)	lateral load (kN)	rotation angle (%)	lateral load (kN)	rotation angle (%)	lateral load (kN)	rotation angle (%)	lateral load (kN)
No.1	0.07	35.4	-0.91	-101	—	—	1.01	106
No.2	0.13	71.2	-1.51	-270	1.15	235	2.02	265
No.3	0.06	55.1	-1.27	-225	0.99	202	2.00	249
No.4	0.10	59.9	-0.89	-186	0.79	176	2.00	213

*calculated from strain gauge

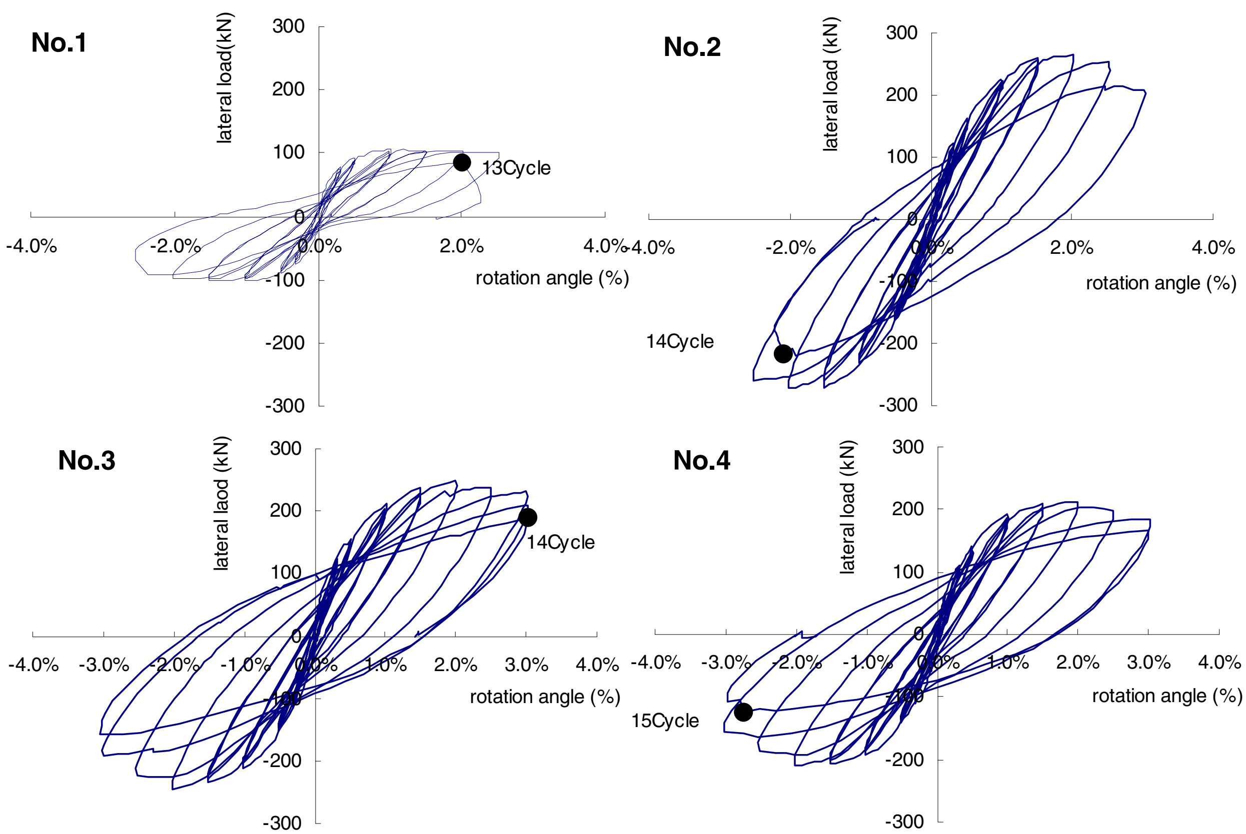
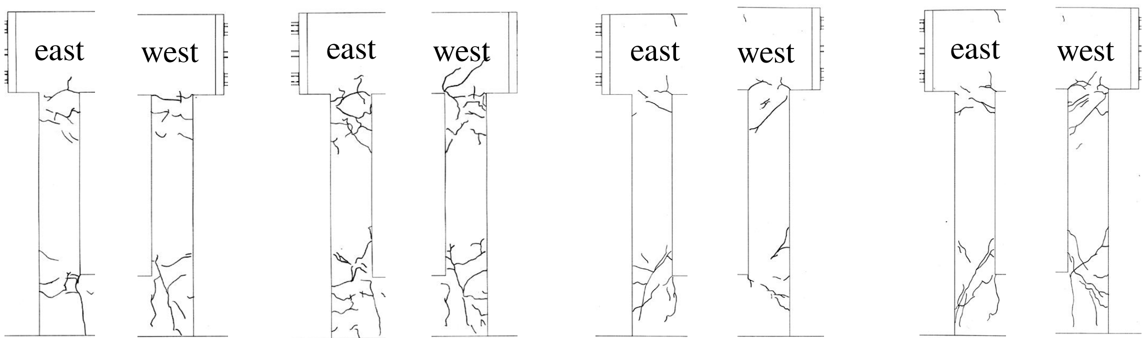


Fig.11. load-deformation relation

Although there were little differences between crack pattern of specimen No.1 and No.2 at 1.0% rotation angle, cracking area of specimen No.1 became wider than that of specimen No.2 at 2.0% rotation angle. Furthermore larger shear cracks were observed at the base of specimen No.1 than that of specimen No.2. Similar tendency was observed in specimen No.3 and No.4.

Early-age cracking of each specimen was observed visually at about the same rotation angle, however, the lateral load of specimen No.2-4 reached 1.5-2 times as large load as specimen No.1 at the point. Furthermore, the rotation angles at the starting point of main reinforcement yielding of RC column became smaller in order from No.2 to No.4. Yielding of main reinforcement in RC column occurred at almost the same deformation level both in the cases of specimen No.1 and No.4. The lateral loads of specimen No.2-4 at the starting point of the yielding reached 1.8-2.7 times as large load as No.1, which was significant increase. For the rotation angle at the starting point of yielding of portal steel frame became smaller in order from No.2 to No.4. The shear cracking load of specimen No.1 was about 104 kN that approximately correspond with the theoretical value as described later in Eq. (4).



No.1(1.0%) No.1(2.0%) No.2(1.0%) No.2(2.0%)

Fig.12. crack of No.1 and No.2

(a) No.1 (b) No.2 (c) No.3 (d) No.4

Photo 1. crack of columns

Connection between steel portal frame and existing RC frame

In the case of reinforced specimens, cracks formed on the boundary surface of RC and non-shrinking mortar injected into the connection of RC frame and steel portal frame, became wider as lateral load increased. However, the non-shrinking mortar did not fall off even at significant deformation level. In the case of specimen No.2, the cracks appeared at about 220kN of lateral load, which is a little lower than the yield strength shown in Table4. At the final cycle, mortar fell out at the base, which did not cause significant drop of lateral load resistance.

Estimation of load-deformation relation

Load-deformation relation after reinforcement is estimated according to the following manner by simply adding theoretical values of RC frame and portal steel frame. Mechanical properties of materials are summarized in Table 2.

load-deformation relation of RC frame

Load-deformation relation of RC frame was determined in the following manner. Upper beam and bottom beam is assumed to be rigid, resulting in shear span to be the internal length between upper and bottom beam.

Flexural cracking moment(Mc) is calculated from Eq.(1) shown in reference[8]

$$M_c = 0.56\sqrt{\sigma_B} \cdot Z_e + ND/6 \qquad (1)$$

Where Z_e=modulus of section, N=axial force.

Additional axial force induced by upset moment when flexural cracking occurred are $\pm 14kN$. Flexural cracking moments calculated taking into account it are 10.4 and 9.5 $kN \cdot m$, and the average of them is adopted because of their little differences. Therefore, the lateral load when RC column reached flexural cracking moment is shown in Eq. (2)

$$Q_{MC} = 2\times\frac{2\times M_c}{h}(=44.3kN) \tag{2}$$

where h is inner length between upper and bottom beam.

Shear cracking strength of RC column is calculated by adding term that represents the contribution of axial force to Eq. (3) that shows shear cracking strength of RC beam. The contribution of axial force is estimated according to the equation indicating shear failure strength of prestressed RC column in reference [9]. Additional axial force induced by upset moment when shear cracking occurred are $\pm 31kN$. Shear cracking moments calculated taking into account it are 47.2 and 46.1 $kN\cdot m$, and the average of them is adopted because of their little differences.

$$_BQ_c = 2\times \mathrm{Ac}\cdot\tau_c = 2Ac\cdot\frac{0.085k_c(500+\sigma_B)}{M/Qd+1.7} \tag{3}$$

$$_cQ_c = 2\times Ac\cdot\tau_c = 2Ac\cdot\frac{0.085k_c(500+\sigma_B+0.1\sigma_0)}{M/Qd+1.7}(=104.7,\ 102.6\ kN) \tag{4}$$

where Ac represents full cross-sectional area of column and beam, k_c represents compensating rate based on section dimension, and σ_0 is axial stress.

Story shear $_cQ_c$ corresponds to a lateral load 103.7kN, which coincides approximately with the experimental value. Ultimate flexural moment is calculated from Eq. (5) in reference (10)

$$M_u = m\times a_t\times\sigma_y\times D + 0.5\times N\times D(1-\frac{N}{bDF_c}) \tag{5}$$

where m is the ratio of the distance between centers of gravity of main reinforcements divided by depth of column, a_t is the cross-sectional area of tensile reinforcement, and σ_y is the yield stress of tensile reinforcement.

Unlike the case of ordinary RC column section where m is defined as 0.8, m is defined as 0.65 by substituting the dimension of specimens for the original form of Eq.(5). Additional axial force induced by upset moment when RC column reached the ultimate flexural moment is $\pm 36kN$. Ultimate flexural moments calculated taking into account it are 28.1 and 23.1$kN\cdot m$. Therefore ultimate flexural strength is calculated from Eq.(6)

$$Q_{Mu} = \frac{2({}_LM_u + {}_RM_u)}{h}(=114.5kN) \tag{6}$$

where $_LMu$, $_RMu$ are the ultimate flexural moments of left and right column respectively.

Ultimate shear strength of RC column is calculated from Eq. (7) in reference [11].

$$Q_{su} = \left\{\frac{0.068p_t^{0.23}(F_c+18)}{M/(Qd)+0.12} + 0.85\sqrt{p_w\cdot\sigma_{wy}} + 0.1\sigma_0\right\}bj \tag{7}$$

where pt, pw, σw, σ_0, and j represent tension reinforcement ratio, shear reinforcement ratio, yield stress of shear reinforcement, axial stress of column, and distance between the centers of tensile and compressive stresses.

Additional axial force induced by upset moment when RC column reached ultimate shear strength is ±39.1kN. Under the assumption that the stress was not redistributed between right and left columns, double amount of the smaller value ,119.2kN, is applied as the ultimate shear strength of RC column. Consequently, the ultimate shear strength of this RC column is defined as Q_{mu}, because Q_{mu} is smaller than Q_{su}.

The secondary modulus after the early-age cracking is calculated from Eq.(8) in reference [12]. Additional axial force induced by upset moment is not taken into account.

$$\alpha_y = (0.043 + 1.64np_t + 0.043\,a/D + 0.33\eta_0)(\frac{d}{D})^2(=0.261) \tag{8}$$

where η_0 is defined as $N/bD\sigma_B$, n, a, and d represent modulus ratio, shear span, and effective depth respectively.

Load-deformation relation obtained from the values above is shown in Fig. 13.

load-deformation relation of reinforced specimens

For reinforced specimens, load-deformation relation was obtained by simply adding the theoretical values of RC frame and steel portal frame. Spiral hoop and non-shrinking mortar injected between the RC frame and steel portal frame are not counted to estimate load-deformation relation.

Maximum story shear of steel portal frame is defined as the smaller lateral load applied to steel portal frame calculated from the material test results when steel column reached full plastic moment or panel zone reached shear yielding. Shear span of steel column was set 1065mm, which is the distance between the bottom of upper beam and the crest of lower beam. The deformation of steel portal column is calculated by adding flexural deformation of column to relative story displacement caused by shear deformation of panel zone.

The contribution of axial load to steel column is defined as 1.0% and 3.0% of yield stress of flange, respectively as described previously or in Table 3. Additional axial force induced by upset moment and the initial axial load are taken into account to calculate full plastic moment. Stress distribution of steel column cross section when steel column reached full plastic moment is shown in Fig.14. Full plastic moment of specimen No.2-4 calculated taking into account these contributions are 42.4, 42.9$kN \cdot m$, 28.8, 29.3$kN \cdot m$, 27.3, 27.6$kN \cdot m$, respectively. Upper beam and bottom beam are assumed to be rigid as well as No.1.

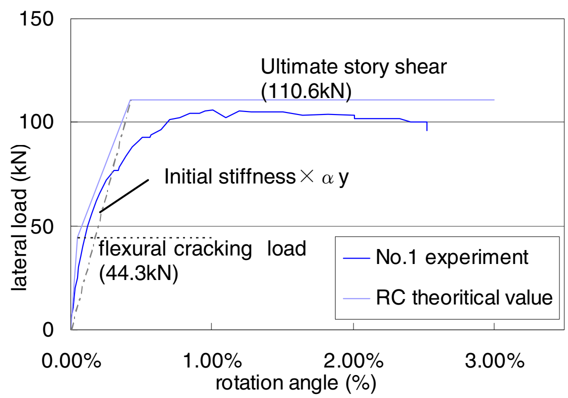


Fig.13. envelope curve of No.1 and RC theoritical value

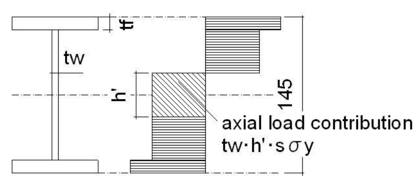


Fig.14. stress distribution

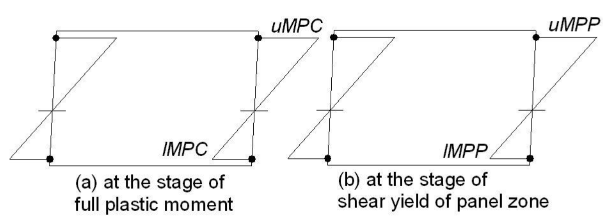


Fig.15. flexural moment of steel column

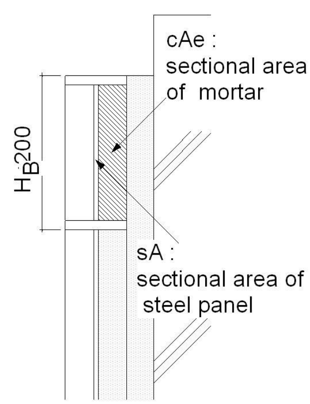


Fig.16. section of panel zone

Story shear of steel column when reached full plastic moment is calculated from Eq.(9) (see Fig. 15(a))

$$Q_{MPC} = \frac{{}_uM_{pc} + {}_lM_{pc}}{h} \qquad (9)$$

where *uMpc* and *lMpc* represent full plastic moment of steel column at column capital and the same moment at column base respectively.

Shear yield strength of panel zone (*Qpp*) is calculated from Eq.(10) in reference [14].

$$Q_{PP} = 0.3F_c \cdot {}_cA_e + \frac{{}_s\sigma_y}{\sqrt{3}} \cdot {}_sA \qquad (10)$$

where *cAe* is sectional area of mortar injected in the steel panel, *sA* is sectional area of steel panel, and ${}_s\sigma_y$ is yield stress of web (see Fig.16). The coefficient of the first term in the right side is set 0.3 on the basis of the assumption that mortar injected to panel zone is so fully bonded by flange and stiffener that it can be taken as under the condition of cross-shaped joint, which is considered valid because there was no remarkable deformation in flanges and stiffeners after the experiment. Therefore, yield moment when shear yielding of panel zone was occurred is calculated from Eq.(11)

$$M_{PP} = Q_{PP} \times (H_c - t_f) \qquad (11)$$

where *Hc* is depth of column, and *tf* is thickness of flange.
Lateral load then is calculated from Eq.(12)

$$Q_{MPP} = \frac{{}_uM_{PP} + {}_lM_{PP}}{h} \qquad (12)$$

where *uMpp* and *lMpp* represent yield moment of panel zone at column capital and the same moment at column base respectively.

Relative story displacement induced by shear displacement of panel zone is calculated in the following manner. Shear stress distributed to panel zone is shown in Fig.17. Shear displacement of panel zone is estimated taking into account non-shrinking mortar injected there. It was assumed that the modulus of rigidity decreases by 30% due to shear crack. Although investigation in the case where the decrease of modulus of transverse elasticity ranges from 0% to 50% are also shown in Fig.19, load-deformation relation is not so different among these cases, resulting in disregard of mortar.
Shear strain of panel zone is calculated from Eq. (13)

$$\gamma = \frac{\tau}{G} = \frac{Q}{GA} = \frac{Q}{G_s \cdot {}_sA + G_m' \cdot {}_cA_e} \qquad (13)$$

where *Q* represents shear strength of panel zone, and *Gm'* represents 0.3 times modulus of rigidity of mortar *Gs* (10.1kN/mm^2 is used as *Gm* in this paper), and *Gs* represents modulus of rigidity of steel of 79*kN/mm2*.

Relative story displacement induced by shear displacement of panel zone is obtained from Eq.(14)

$$\delta_{PN} = \frac{{}_u\gamma + {}_l\gamma}{2} \cdot h \qquad (14)$$

where ${}_u\gamma$ and ${}_l\gamma$ represent shear strains at column capital and column base respectively.

Stress distribution of steel portal frame is shown in Fig.18 where the inflection point of elastic zone is determined from the smaller value of full plastic moment at both ends of column and shear yield moment of panel.

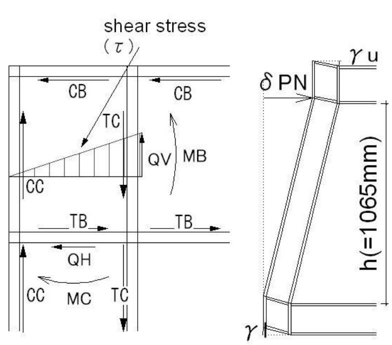


Fig.17. shear force of panel zone

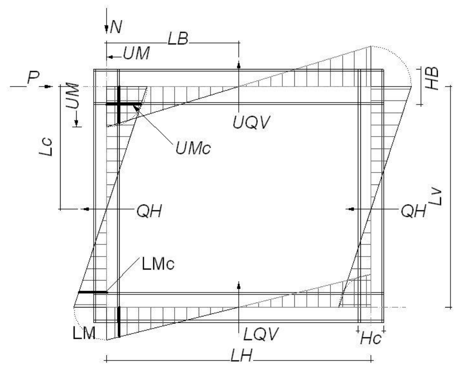


Fig.18. stress of steel frame

Axial force(*C,T*) which is assumed to be distributed to only steel flange are represented from Eq.(15).

$$T_c = |C_c| = \frac{M_c}{H_c - t_f} \qquad (15)$$

where *Mc* is flexural moment applied to both ends of column.

Average shear force distributed in panel zone is obtained by subtracting member shear force (*Qv*) from *Tc*, because *Qv* acts in the reverse direction of *Tc*. In the case of *L* shaped connection as this experiment, it is assumed that *Qv* is distributed in a triangle as shown in Fig. 17, and average shear force is represented from Eq. (16).

$$Q = T_c - Q_v / 2 \qquad (16)$$

Ultimate story shear and displacement of specimen No.2-4 obtained from values above are summarized in Table5. Comparison between theoretical value and experimental value of load-deformation relation of specimen No.2 is shown in Fig.19., which indicates that strength is estimated conservatively.

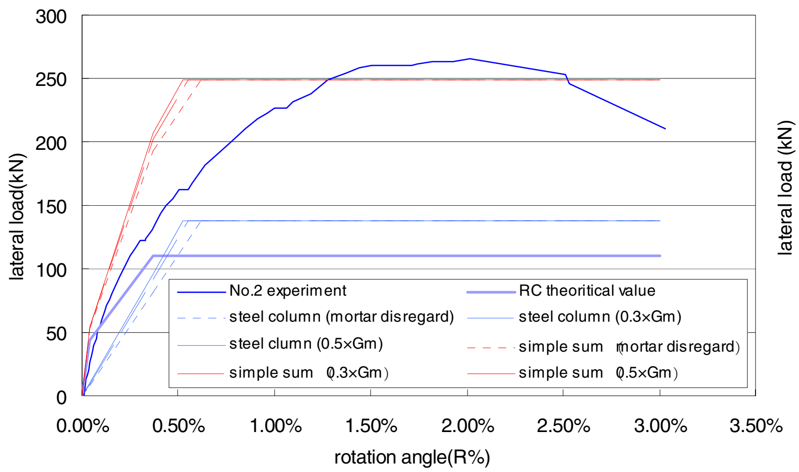


Fig.19. No.2 experimental value and theoretical value

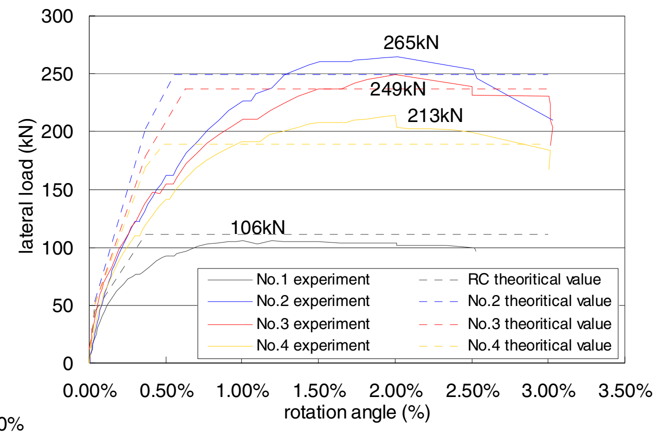


Fig.20. comparison between experiment and theoretical value

Comparison between experimental value and theoretical value of load-deformation relation

Comparison between experimental value and theoretical value of load-deformation relation is shown in Fig.20. Ultimate strengths of reinforced specimens are estimated on the safe side by simply adding theoretical strength of RC frame to steel portal frame. Furthermore, theoretical ultimate shear strength of each specimen is as well or a little higher than the load at the beginning point of steel frame yielding.

The ratio of relative story displacement induced by shear deformation of panel zone to total displacement at the point of ultimate story shear ranges 40% to 50% as summarized in Table5. It is indicated that reinforcement of panel zone is necessary in order to improve the effectiveness of reinforcement by means of steel portal frame.

Table 5. ultimate story shear and relative story displacement os steel frame

	ultimate story shear		relative story displacement		
	at the stage of full plastic moment Eq.(7)	at the atage of shear yield of panel zone Eq. (10)	flexural deformation	caused by shear deformation of panel zone	total
No.2	80.9	○ 69.1	3.11	3.46	6.31
No.3	○ 63.0	68.2	3.66	3.98	7.26
No.4	○ 39.0	71.9	3.68	2.25	5.71

CONCLUSION

One-third scale RC frame specimens with single-story single-span that represent flexure-dominant column of RC building with five-six-story were tested under the cyclic loading in order to confirm the effectiveness of PG Method. Conclusion obtained from the experiment is itemized below.

1. Reinforced specimens (No.2-4) by means of steel portal frame behave stable compared to non-retrofit specimen (No.1) under cyclic lateral loading. Furthermore, significant drop of load resistance did not occur after the shear failure of RC column in No.2-4, although shear failure at column base induced significant drop of lateral load resistance in No.1.
2. The maximum load of No.2-4 advanced by 2-2.5 times as that of No.1 that indicates full effectiveness of strength reinforcement.
3. Strength obtained by simply adding theoretical value of RC frame to steel portal frame estimates experimental value on the safe side.

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