SEISMIC BEHAVIOR OF HYBRID SYSTEM WITH CORRUGATED STEEL SHEAR PANEL AND RC FRAME

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ABSTRACT :

This research aims to propose an economical seismic response controlling system of RC frames using corrugated steel shear panels (CSSP). This hybrid system was originally proposed by Mo and Perng in 2000. In this study, their system was revised and new hybrid system was proposed to prove that the shear capacity and stiffness of CSSPs can be fully utilized if sufficient anchorage is provided. In an experimental phase, a stud-type anchorage often used in bridge box girders was employed. The behavior was stable if the number of studs satisfied the Japanese design guidelines. The behavior after buckling of CSSP was ductile and degradation in lateral load carrying capacity was about 20% even at 5% story drift angle. However, even specimens with half number of studs showed the similar behavior. All hybrid frames showed more than 30% increase in lateral load carrying capacity as originally designed. In an analytical phase, the analytical model proposed well simulated the behaviors of frames with different anchorage of the shear panels, considering the effective area of CSSP which carries the lateral load. The analytical result showed that CSSP carries more than 90% of its shear capacity even if the panel has the half number of studs determined by the design guidelines. This study proved that CSSP has a potential to make a main structural component to carry lateral load. The use of CSSP in building structures has just begun in Japan.

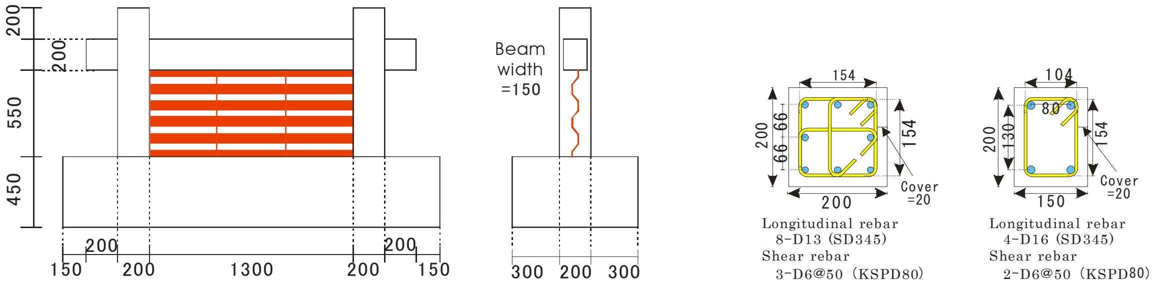
KEYWORDS: Corrugated steel shear panel, Improvement of ductility, Damage control, Energy dissipation, Frame analysis

1. INTRODUCTION

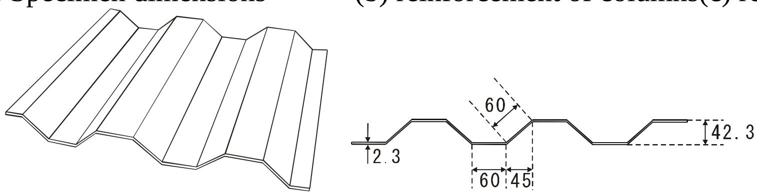
It is a common practice to use reinforced concrete shear walls in reinforced concrete structures to maintain high lateral load carrying capacity and stiffness. However, high lateral stiffness with brittle ultimate failure mode of RC structural walls often requires high lateral load carrying capacity according to their seismic response characteristics. In order to improve the ductility of reinforced concrete shear walls, some efforts have been made such as using low yield strength reinforcement and introducing slits but the ductility enhancement was not very prominent. Use of steel shear walls in order to increase ductility has some decades of research history. In 1973, Takahashi et al. (1973) studied the characteristics of load-deflection relations of steel shear walls obtained experimentally and reported the effects of configuration, width-thickness ratio, stiffeners' stiffness, etc. on the load-deflection relations. Studies on steel shear walls have been continued since then by Gaccese V (1993) and Driver (1998). However, flat steel shear panels need stiffeners to prevent plate buckling, leading to the increase of self-weight and cost. In order to solve these problems, corrugated steel shear panels have been used in bridge structures since 1990's. They weigh less and decease prestressing loss due to their negligible axial stiffness compared to flat steel shear panels reinforced with ribs. In 2000, Mo and Perng (2000) reported a use of corrugated steel shear panels as a main lateral load carrying component for building structures. They proved that corrugated steel shear panels are effective to delay buckling of shear panels. However, bolt anchorage fastening the shear panel to the surrounding RC frame was not very effective and a large slip took place at the interface resulting in pinched hysteresis loops with small energy dissipation. Their test results provided interesting information on the potential use of corrugated steel shear panels but shear panels have not been used in practice as a main lateral load carrying component. This paper proposes a use of corrugated steel shear panels for shear walls instead of reinforced concrete shear walls by introducing the experimental work on RC portal frames with corrugated steel shear panels. The stud-type anchorage of the shear panel used in bridge girders was employed and the shear capacity and stiffness of shear panels were fully utilized. Corrugated steel shear panels have larger buckling strength than the flat panel due to its configuration, with negligible flexural and axial stiffness. They dissipated much greater energy after the peak load compared to RC shear walls. Employing them as a main lateral load carrying component in building structures makes it possible to assign vertical load to columns and shear load to corrugated steel shear panels, resulting in a clear design philosophy. In addition, the ductility after shear yielding or even after buckling is excellent and the required lateral load carrying capacity may be decreased by considering the equal energy dissipation theory.

2. EXPERIMENTAL SETUP

Specimens were made of reinforced concrete portal frame with different anchorage configurations of corrugated steel shear panels. Dimensions of four RC frames are identical as shown in Figure 1 and test variables are shown in Table 2.1. All shear panels had flange at the four side and two vertical stiffeners as shown in Figure 2. Thickness of flange at vertical sides of Specimen C and four sides of Specimen D was 9 mm whereas other flange and stiffeners were 4.5mm. Mechanical properties of materials are listed in Table 2.2.



(a) Specimen dimensions (b) reinforcement of columns (c) reinforcement of a beam

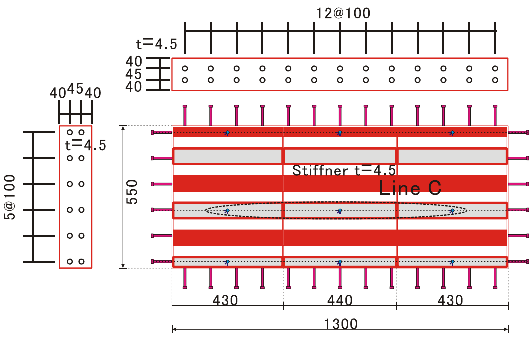


(d) Dimensions of the corrugated steel shear panel used in the experiment.
Figure 1 Dimensions and reinforcement arrangement of specimens (Unit:mm)

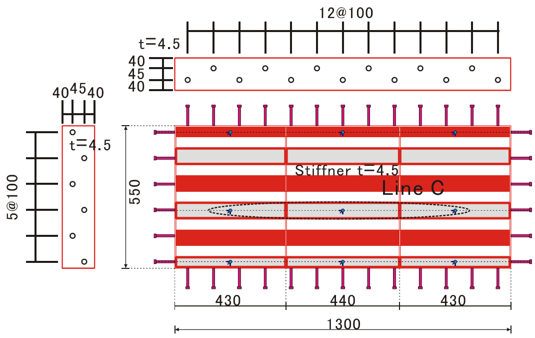
Table 2.1 Test variables

Specimen	Anchorage of the corrugated shear panel to the surrounding frame	Arrangement of studs	
		Horizontal joints (No. of studs)	Vertical joints (No. of studs)
A	studs	ϕ9 double@100 (26)	ϕ9 double@100 (12)
B		ϕ9 staggered@100 (13)	ϕ9 staggered@100 (6)
C		ϕ9 double@100 (30)*	None
D	grout mortar	None	None

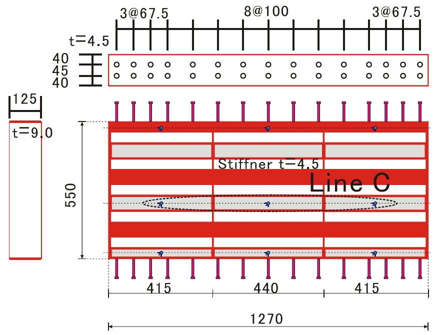
* ϕ9 double@67.5 was used at the end region.



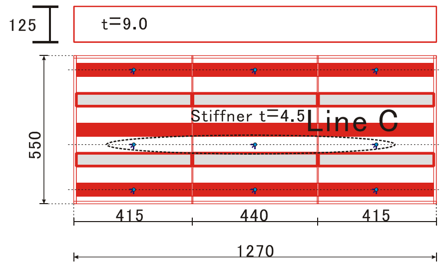
(a) A (Double studs)



(b) B (Staggered studs)



(c) C (Double studs on two sides)



(d) D (No stud)

Figure 2 Dimensions of shear panels (Unit:mm. All studs had a 9mm-diameter bolt with a head)

Table 2.2 Mechanical properties of materials

(a) Concrete

	Compressive strength (MPa)	Tensile strength (MPa)	Young's modulus (GPa)
Concrete	62.0	3.93	29.3
Mortar	63.4	–	–

(b) Steel

Type	Yield strength (MPa)	Tensile strength (MPa)	Young's modulus (GPa)
D6	1099	1207	196
D13	391	551	186
D16	391	569	180
Corrugated panel	264	362	191
Flange plate	282	438	200
Stud	479	512	208

The number of studs of Specimen A was determined using Eqn. 2.1 and the number of studs was halved in Specimen B. The number of studs at the horizontal joints in Specimen C was basically the same with Specimen A but the number was increased so that the spacing was 67.5 mm instead of 100 mm at the end portion to prevent the local failure of studs as shown in Figure 2(c). Specimen D had no stud anchorage but the peripheral spacing was filled with high-strength grout mortal.

$$(p/p_a)^{5/3} + (q/q_a)^{5/3} \le 1.0 \qquad (2.1)$$

Where p is the design tension force, q is the design shear force, p_a is the tensile strength when the stud experiences tension only, q_a is the shear strength when the stud experiences shear force only. The tensile strength, p_a, is the minimum value of 1) tensile strength due to a cone failure of surrounding concrete, 2) tensile strength due to tensile yielding of the stud, and 3) tensile strength due to bearing failure of concrete. The shear strength, q_a, is the minimum value of 1) shear strength due to bearing failure of concrete, and 2) shear strength due to shear yielding of the stud. The design tensile force, p, and the design shear force, q, was obtained from elastic FEM analysis as shown in Figure 3. In an analytical model, two columns and a beam consisted of cubic solid elements and corrugated shear panel and other steel plates consisted of shell elements. When the shear panel reached the yielding strength, the maximum normal stress was 31.5 N/mm^2 and the average shear stress was 160 N/mm^2 at the upper edge of the shear panel, which were substituted in p and q. Using of double studs of ϕ 9 at 100 mm spacing, the left side of Eqn. 2.1 became 0.99 and the equation was just satisfied. This determined the number of studs at the upper horizontal joint of Specimen A. The other interfaces were similarly computed.

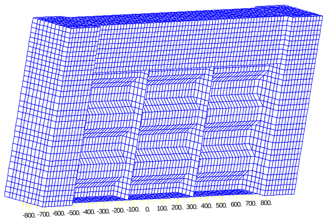


Figure 3 FEM analytical model to find the stresses at the interface between the shear panel and concrete.

Figure 4 shows the loading system. Constant axial load of 365 kN (Axial load level 0.15) was introduced to each column. Equal magnitude of lateral load was applied to the both ends of the beam by two 1000 kN hydraulic jacks. Two cycles of lateral load was applied at ±150 kN and ±250 kN. Then two cycles of preselected drift was enforced at ±0.1%, ±0.2%, ±0.4%, ±0.6%, ±0.8%, ±1.0%, ±2.0%, ±4.0%. After 4.0%, drift was monotonically increased to +10 % which was the limit of the loading system.

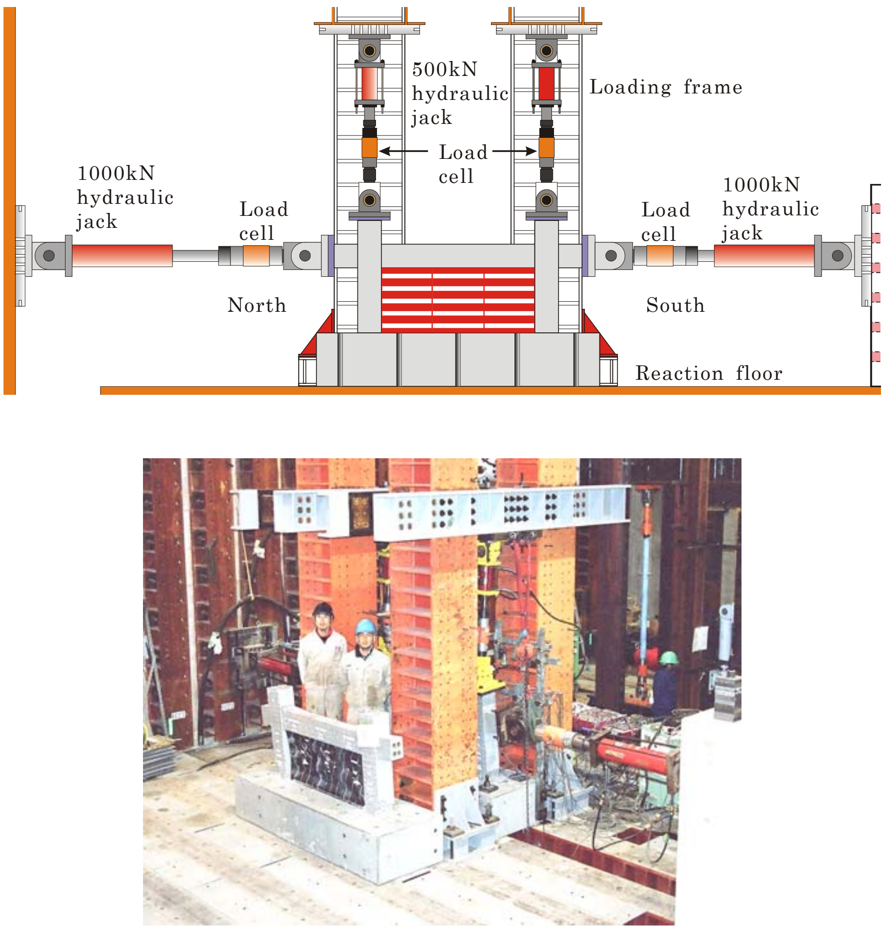


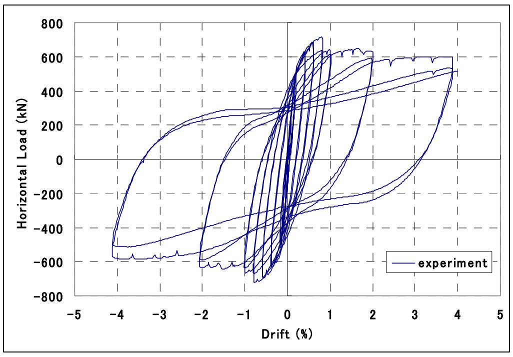
Figure 4 Loading system

3 EXPERIMENTAL RESULTS

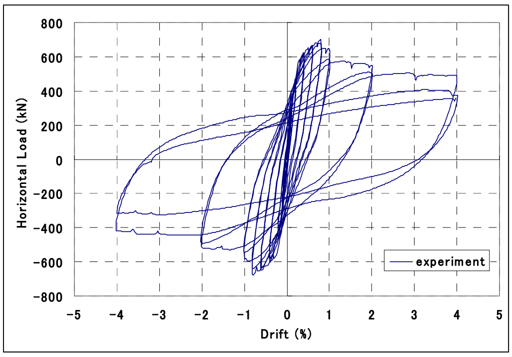
3.1. Lateral load– drift relations

Figure 5 shows the lateral load – drift relations up to R=4.0%. Specimens A, B, and C showed similarly fat hysteresis loops up to the peak load at which buckling took place. Even after the buckling, the degradation of load carrying capacity was not drastic as RC shear walls failing in shear and reasonable amount of energy was dissipated. The degradation of load carrying capacity was severe in the order of Specimen C, B, A. Specimen D showed less peak load and hysteresis loops were pinched. Specimen D necessitated large drift angle to reach the peak load and the degradation of load carrying capacity after the peak was smaller than other three specimens. This is because Specimen D experienced openings at the interface between steel flange and joint mortar near the corner after R=±0.2% leading to the slip-type hysteresis loops afterward.

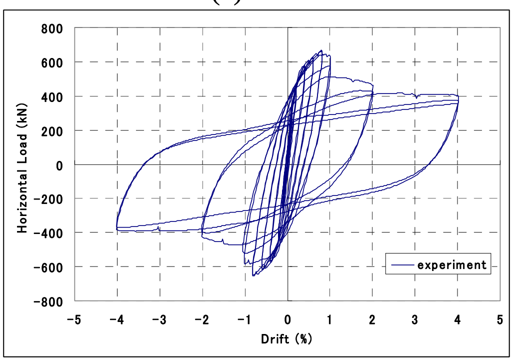
The yielding lateral loads, lateral load carrying capacities and the initial stiffnesses are summarized in Table 3.1. The maximum lateral load capacity, Qmax, caused by buckling of the shear panel in positive and negative directions are in the order of Specimen A, B, C, and D and reflects the number of studs. However, the yielding lateral load, Qy, was similar for all specimens although Qy of Specimen B is slightly higher than the others. Drift angles at yielding, Ry, of Specimen A was the smallest and that of Specimen D was largest. This reflects the number of studs but the initial stiffness does not necessarily reflect the number of studs. The drift angles at the maximum capacity were similar for each other for Specimens A, B, and C but those were much larger than that for Specimen D because of the slip at the interface. Specimens A, B, and C did not show any brittle failure until R=10%. Specimen D showed a large amount of the out-of-plane deformation of the shear panel at R=5.0% and loading was terminated. It can be seen that behavior of the hybrid system is greatly affected by the amount of studs and the resulting constraint.



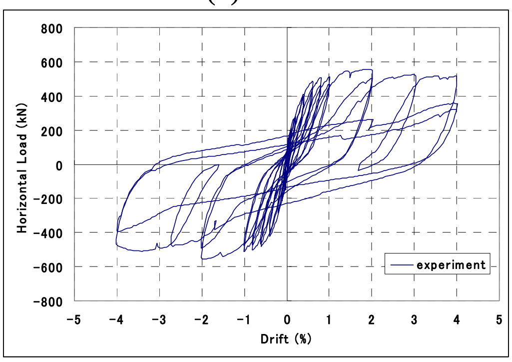
(a) A



(b) B



(c) C



(d) D

Figure 5 Lateral load - drift angle relations

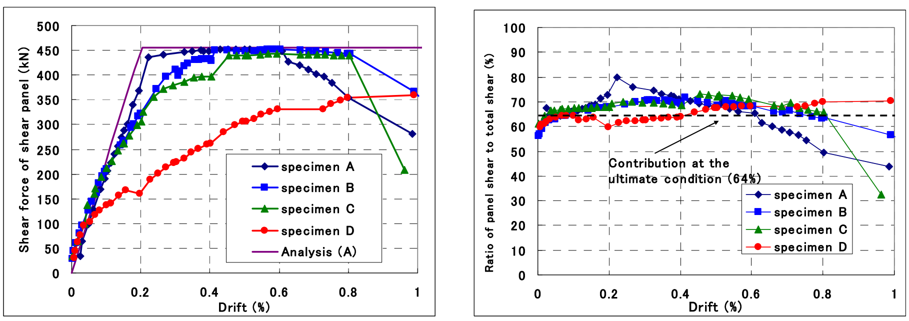
Table 3.1 Summary of test results

Specimen	Yielding lateral load		Maximum lateral load capacity				Initial stiffness (10^5 kN/rad)
			Positive direction		Negative direction		
	Ry (%)	Qy (kN)	R (%)	Qmax (kN)	R (%)	Qmax (kN)	
A	+0.222	546	0.801	716	−0.759	−720	5.55
B	+0.369	614	0.797	702	−0.803	−676	4.10
C	+0.337	549	0.803	667	−0.783	−655	3.44
D	+1.388	544	1.97	556	−1.94	−555	4.56

3.2. Lateral load carried by shear panel

Lateral load carried by the shear panel is plotted in Figure 6 (a). Shear force of the shear panel increased rapidly for Specimen A but with slower rate for the other specimens. As the number of studs increased, the shear panel became stiffer and the buckling initiated earlier. The ratio of lateral load carried by the shear panel to the total lateral load is plotted in Figure 6 (b) up to R=1.0%. The shear force carried by shear panel was computed from three Rosetta strain gages on Line C in Figure 2 assuming the plane stress and elastic-perfectly plastic yield condition with the von Mises yield criteria. It is seen that the shear panel carried 60% to 70% of the lateral load from the very beginning of the loading till buckling took place at R=1.0%. The computed contribution was expected to be 64.0% at the ultimate condition by considering the story shear force at the formation of collapse mechanism of the surrounding RC frame and the shear force of the shear panel at yielding.

Equivalent viscous damping ratio, Heq, is shown in Figure 7. Heq of specimens with studs (Specimens A, B and C) increased rapidly from R=0.4% at which the shear panel yielded, and a large amount of energy was dissipated even after the buckling. Specimen A had largest Heq and Specimen B had the second largest Heq until yielding. Even after R=1.0%, a large amount of energy was dissipated in Specimens A, B, and C. The shear panel of Specimen D did not follow the deformation of the RC frame and the yielding of the shear panel was delayed leading much smaller energy dissipation and Heq.



(a) Shear force of shear panel (b) Ratio of lateral load carried by the shear panel
Figure 6 Lateral load carried by the corrugated shear panel

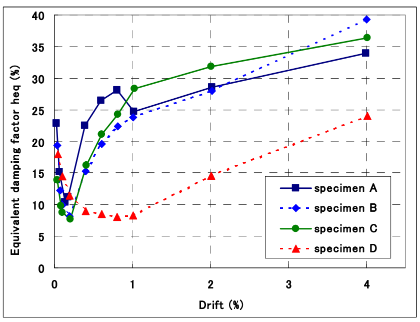
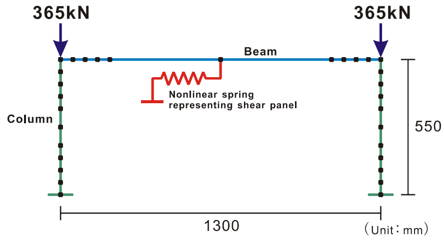


Figure 7 Equivalent viscous damping ratio, H_{eq}

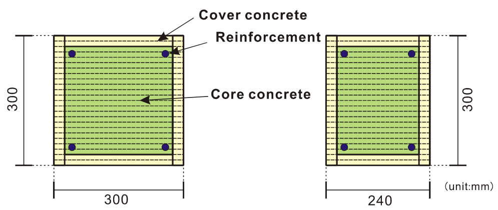
4 ANALYTICAL MODELING

4.1. Numerical model with a frame analysis program

Behavior of the RC frame with the corrugated shear panel was simulated using a frame analysis program. The analytical model is shown in Figure 8. Two columns and a beam were modeled as a line element as shown in Figure 8(a). Each line element consisted of plain concrete fibers, confined concrete fibers, and reinforcement fibers as shown in Figure 8(b). Since the corrugated steel shear panel had shear stiffness without either axial or flexural stiffnesses, it was modeled with a nonlinear spring with an equivalent stiffness in the axial direction to the shear stiffness of the shear panel. The center of the beam is connected to the ground through this spring. Considering the slip and opening behavior between the shear panel and surrounding RC frame, effective area of horizontal section which carries the lateral load was decreased according to the amount of the studs for Specimens B and C.



(a) Model

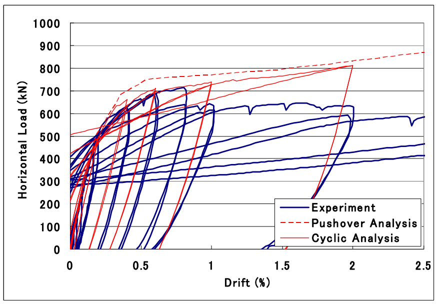


(b) Columns and beam section

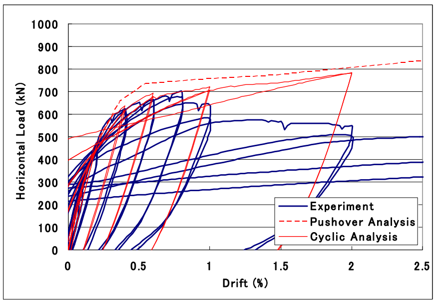
Figure 8 Analytical model

4.2. Analytical results

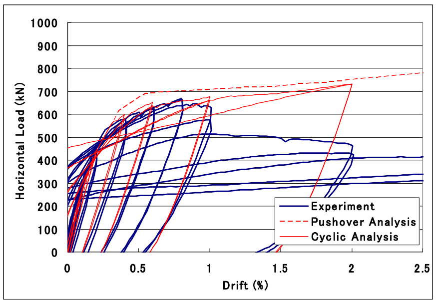
Results of pushover and cyclic analysis are shown in Figure 9. Cyclic analysis results well simulated the experimental loop for Specimen A up to the peak load at which buckling of the shear panel took place. After buckling, experimental loop became pinched but the analysis didn't show this degradation. For Specimens B and C, hysteresis loops were well simulated until R=0.8% when the buckling occurred at the shear panel if effective areas in carrying lateral load were decreased to 97% and 90% of the whole horizontal section of the shear panel. Pushover analysis was slightly higher than the envelop curve of the experimental loop for Specimens A, B and C. Although the effective area of the shear panel which carries the lateral load needs to be formulated by collecting more data, the analytical result showed that the shear panel carried more than 90% of its shear capacity even if the panel had the half number of studs determined by the design guidelines (AIJ 1985).



(a) A



(b) B



(c) C

Figure 9 Analytical results

5 CONCLUSIONS

Four 40% scale RC frames reinforced with the corrugated steel shear panel were loaded statically to see the effect of anchorage studs on the seismic performance of the shear panel, and the analytical model to simulate the frame behavior was proposed.

- The revised hybrid system with corrugated steel shear panels excellently behaved as a seismic controlling system with large shear stiffness and shear capacity. In addition, the system showed some increase in shear force after yielding until buckling. The behavior after buckling was ductile and degradation in lateral load carrying capacity was about 20% of the peak load even at R=5%. The behavior was stable if the number of studs satisfied the Japanese design guidelines (Specimen A). However, even specimens with half number of studs (Specimens B and C) showed the similar behavior although the stiffness and lateral load carrying capacity was slightly lower.
- Lateral load carrying capacity at the peak was greater and the post-peak degradation in lateral load carrying capacity was greater for specimens with the larger number of studs. Specimen D, which had no stud anchorage, had smaller lateral load carrying capacity.
- Corrugated shear panel dissipated large amount of energy after yielding and the dissipation continued even after buckling of shear panel. Specimen D had inferior behavior on energy dissipation because of slip and opening at the interface between the shear panel and the peripheral frame.
- Considering the effective area of shear panel which carries the lateral load, analytical model well simulated the behaviors of Specimens A, B and C. The analytical result showed that the shear panel carries more than 90% of its shear capacity even if the panel has the half number of studs determined by the design guidelines.

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