SEISMIC PERFORMANCE OF ISOLATED NONSTRUCTURAL CONCRETE PARTITIONS WITH DOWEL-BAR CONNECTORS UNDER CYCLIC LATERAL LOAD

H. Shiohara[1], K. Okubo[2], Zhou-yi Chen[3] and F. Kusuhara[4]

[1] *Associate Professor, School of Engineering, The University of Tokyo, Tokyo, Japan*
[2] *Research Engineer, Institute of Technology, Shimizu Corporation, Tokyo, Japan*
[3] *Research Student, School of Engineering, The University of Tokyo, Tokyo, Japan*
[4] *Assistant Professor, School of Engineering, The University of Tokyo, Tokyo, Japan*
Email: shiohara@arch.t.u-tokyo.ac.jp, kaori_okubo@shimz.co.jp, zychen_1999@yahoo.com.cn and kusuhara@rcs.arch.t.u-tokyo.ac.jp

ABSTRACT :

Nine full-scale isolated non-structural reinforced concrete panels with in-plane dowel connectors jointed to the surrounding structural frames were tested under cyclic displacement-controlled seismic loading to quantify the strength and the ductility of the panels and to assess the damage of the non-structural partitions. The specimens were installed in a steel loading frame of lateral resistance free and were loaded until the fracture of dowel connectors which caused significant reduction of lateral resistance. The test parameters include: (a) panel dimensions, (b) amount and arrangement of the dowel connectors, and (c) gap distance between panel and frame. Test result indicates that the precast concrete panels could provide strength increase of the structural system, desirable deformation capacity and additional energy dissipation capability if appropriate dowel-bar connecting details were provided.

KEYWORDS: dowel connector, isolated non-structural partition, seismic performance

1. INTRODUCTION

Non-structural partitions are used to provide buildings of moment resisting frame structural system with shelter and separate of space. Isolated precast concrete or isolated cast-in-place concrete panels are effective in reinforced concrete buildings to reduce the seismic damage of the nonstructural partitions as well as to avoid captive column effect and complicated structural modeling of the non-structural elements in seismic design. But non-structural partitions are connected to the surrounding frames by connectors to keep the position and to support out of plane lateral force. In spite of the resistance of the connectors to in-plane action, the seismic behavior of such connectors has received little attention. They are always neglected for assessment of seismic performance of the structural system. Among the various connectors, dowel deformed rebars are preferred due to the low cost and simple construction in Japan. But there are no design guidelines for the partition jointed to surrounding frame. Some studies reported on the dowel action for reinforcing bars in concrete members (Syam S. Mannava et al. 1999; William G. Davids et al. 1997; Vintzeleou et al. 1987). But they primarily focus on the strength of shear transfer in beam elements or the behavior of a dowel across a construction joint. Some of the authors have tested precast concrete panel jointed with dowels (Darama, 2006; Okubo et al. 2006). It has been shown that the precast concrete panel with dowel connectors could dissipate energy if good constructional details were given. This paper reports the result of another tests carried out in 2007 and 2008. Nine full scale reinforced concrete partitions with dowel connectors were tested under cyclic lateral loading. The test parameters include: (a) panel size, (b) amount and arrangement of the dowel connectors, and (c) gap distance between panel and frame.

2. EXPERIMENTAL PROGRAM

2.1. Test specimens

The dimensions, material properties, and reinforcing details of the specimens are chosen from a prototype residential building in Japan. Non-structural cast-in-place reinforced concrete partitions are made with gap of 20mm at the top and the two vertical sides. There is no gap at the bottom of the partition because such partitions are constructed monolithically constructed with the beam. Design strength of concrete is 24 MPa. Table 1 lists the test parameters, which includes: (a) panel size, (b) gap distance between panel and frame, and (c) type of the dowel bar used in connectors.

Table 1 Test parameters

Specimen	Panel size in mm height, width and thickness	Gap distance			Dowel bar type		
		Side mm	Top mm	Bottom mm	Side	Top	Bottom
D1	2000×1200×120	---	20	---	---	ϕ14-ELCH2	D16
D2	2000×1200×120	20	20	---	ϕ14-ELCH2	ϕ14-ELCH2	D16
D3	2000×1200×120	---	20	---	---	ϕ14-ELCH2	D16
D4	2000×1200×120	---	20	---	---	ϕ14-ELCH2	D16
E1	2000×1800×120	---	20	---	---	ϕ14-ELCH2	D16
E2	2000×1800×120	---	20	---	---	ϕ14-ELCH2	D16
F1	2000×600×120	20	20	20	ϕ14-ELCH2	ϕ14-ELCH2	ϕ14-ELCH2
NS1	2000×1200×120	---	20	---	---	ϕ16-ELCH2	D13
S1	2000×1200×120	20	20	---	ϕ12-ELCH2	ϕ12-ELCH2	D13

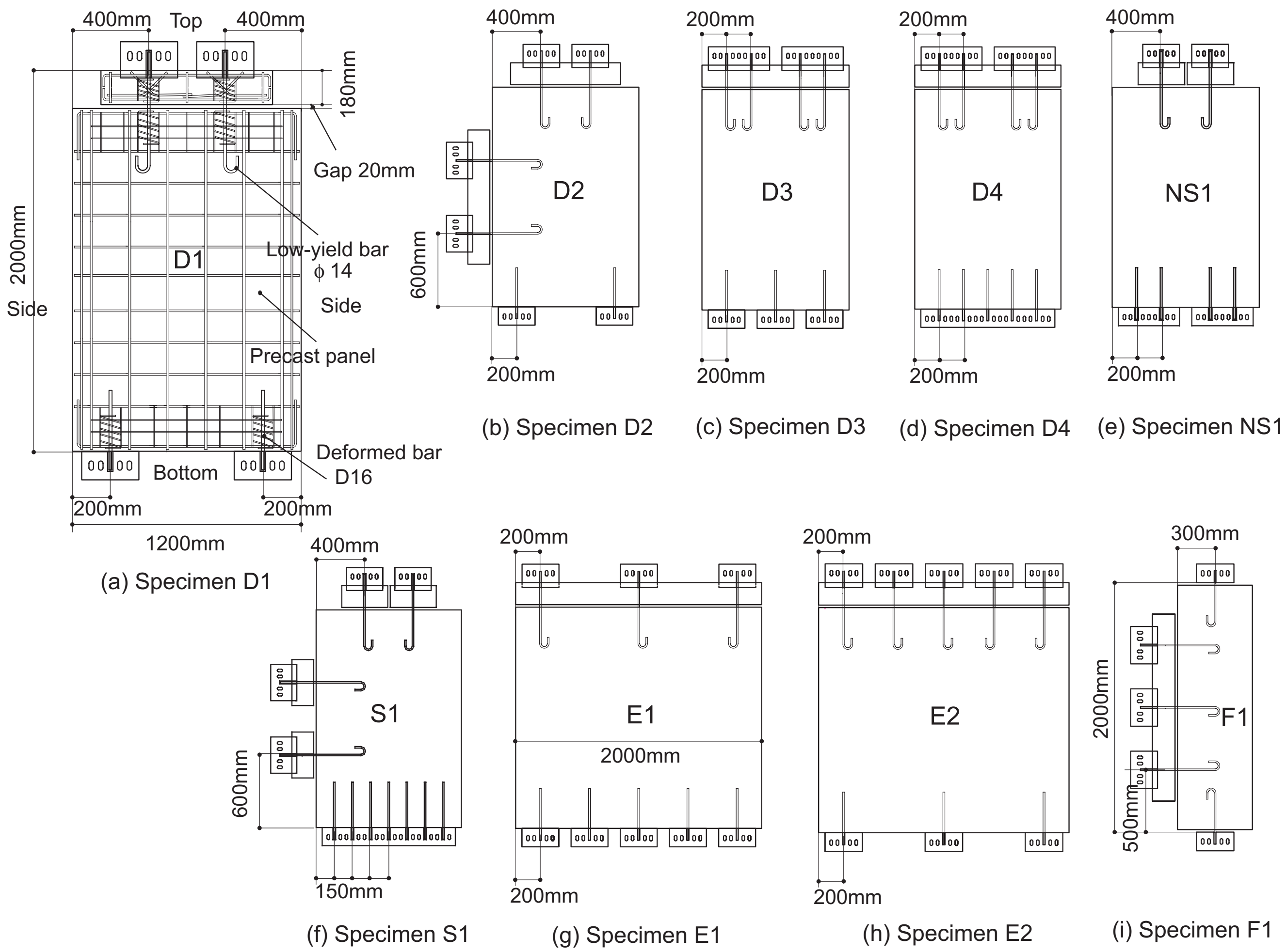

Fig. 1 Geometry and reinforcing detail of specimens

Figure 1 shows the geometry and reinforcing detail of the specimens. For the dowel connectors, plain bars with low yield point steel (ELCH2) were used to obtain high ductility. The specimens were connected to the loading frame shown in Fig. 2 by steel plates welded to the end of dowel bars with holes and high tension bolts. For all the specimens, the reinforcement details in the concrete panels and the connectors were identical except for that of NS1, which has no spiral reinforcements around the dowel bars.

2.2. Mechanical properties of materials

The test specimens were cast in a horizontal position in one batch of concrete in the year 2007 and 2008 respectively. Mechanical properties of concrete and steel bars are listed in Table 2 and Table 3 respectively.

Table 2 Concrete properties

Specimen	Cylinder size	Age of concrete	Cylinder compressive strength	Splitting tensile strength	Elasticity modulus
	mm	Days	MPa	MPa	GPa
D1, D2, D3, D4, E1, E2, F1	100×200	42 85	33.2 36.1	2.78 2.82	30.8 29.7
NS1, S1	100×200	102 130	30.4 35.0	2.50 2.95	26.7 29.9

Table 3 Reinforcement properties

Specimen	Type	Elasticity modulus	Yield strength	Ultimate strength	Fracture strain
		GPa	MPa	MPa	με
D1, D2, D3, D4, E1, E2, F1	D10 (SD345)	225	394	564	0.167
	D13 (SD345)	256	339	557	0.164
	D16 (SD345)	227	402	592	0.168
	Φ14-ELCH2	208	197	301	0.312
NS1, S1	D13 (SD345)	169	341	501	0.173
	Φ12-ELCH2	202	171	287	0.315
	Φ16-ELCH2	209	187	288	0.284

2.3. Test setup

Test setup is shown in Fig. 2. The loading frame consists of four separate steel beams with H-section connected to each other by frictionless pin joints. There is a frame (not shown in Fig.2) to provide the out of plane stability

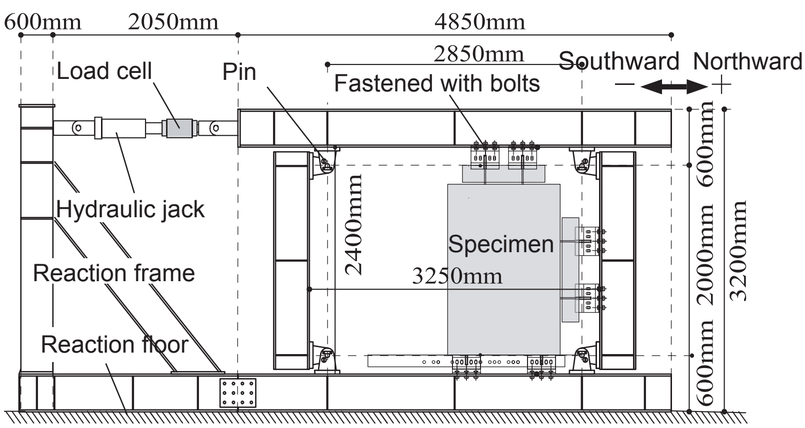

Fig. 2 Test setup

for the loading frame. This loading frame permits an inter-story displacement input between the upper and bottom beams. The upper steel beam was loaded horizontally by a hydraulic jack. The jack was controlled manually to follow the prescribed displacement histories. The story shear was measured by the load cell installed between the hydraulic jack and the horizontal steel beam. Lateral displacement of the upper horizontal beam was measured by a displacement transducer. With lateral displacement being divided by the vertical distance between two pin joints in the loading frame, story drift can be obtained. Other measurements include: relative displacements between panel and loading frame at each connector, displacements of test panel at corners.

2.4. Loading sequence

Each specimen was subjected to cyclic shear loads. The sequence of loading consisted of sets of three cycles. The specified amplitudes were: ±2.5mm、±5mm、±10mm、±15mm、±20mm、±25mm、±30mm、±35mm or until the fracture of connector cause significant loss of lateral resistance of the panel.

3. TEST RESULTS

Strengths, drifts and stiffness are summarized in Table 4. All the nine specimens failed due to the fracture of dowel bars at the connectors. Relations between lateral load and displacement for all specimens are showed in Fig.3.

Table 4 Test results

Specimen name	Elastic stiffness (kN/mm)	Maximum shear strength before drift reach 1/200 (kN)		First fracture of dowel	
		+	–	Shear(kN)	Drift(%)
D1	13.8	21.84	-21.80	14.20	1.25%
D2	14.1	45.98	-54.99	40.04	1.25%
D3	16.7	43.35	-41.44	14.02	1.75%
D4	12.3	41.29	-44.04	-24.43	1.75%
E1	29.1	35.41	-32.73	-15.87	1.5%
E2	30.8	55.65	-55.51	9.88	1.5%
F1	1.90	7.33	-8.09	1.68	4.5%
NS1	15.7	31.40	-34.58	11.80	1.5%
S1	20.5	33.67	-44.03	18.48	1.5%

3.1. Failure Process

Failure of the Specimens D1, D3, D4, E1, E2, and NS1 was similar and characterized by the fracture of the connector of dowel bars at the top side. Photo of a connector of Specimen E1 are shown in Fig. 4. Failure of Specimen S1 occurred on the upper connectors as well as top connectors. The bottom connectors had no distinct damage for all these specimens. There was also no crack found in both face of the panel, except on the side surface, where much of the concrete around the dowel bars of connectors was spalled off. Maximum drifts reached before specimen failed for all seven specimens were almost 1.5% (30 mm). Observed relation of shear and story drift were characterized with slip and uplift (rocking) displacements. It is also noted that the panels tilted out of plane displacement and torsion when closed to failure. However, they are usually very small before the drifts reached 1.0%.

Contrary to the other specimens, Specimen F1 showed no distinct damage to the dowel bars of upper and bottom connectors but side connectors fractured. There were no cracks found in both faces of the panel, whereas much of the concrete around the dowel bars of connectors was shaved off at the side surfaces. Maximum drift reached was almost 4.5% (3.54 in. [90 mm]) before three side connectors dowel bars fractured.

Specimen D2 showed failure at the bottom connectors, accompanied by cracks running through the both bottom corners on the face plane of the panel. Maximum drift reached was almost 1.25% (0.98 in. [25 mm]) before both bottom connectors dowel bars fractured. The in-plane response of panel for D2 was quite large, accompanied by somewhat out of plane displacement and torsion when closed to failure.

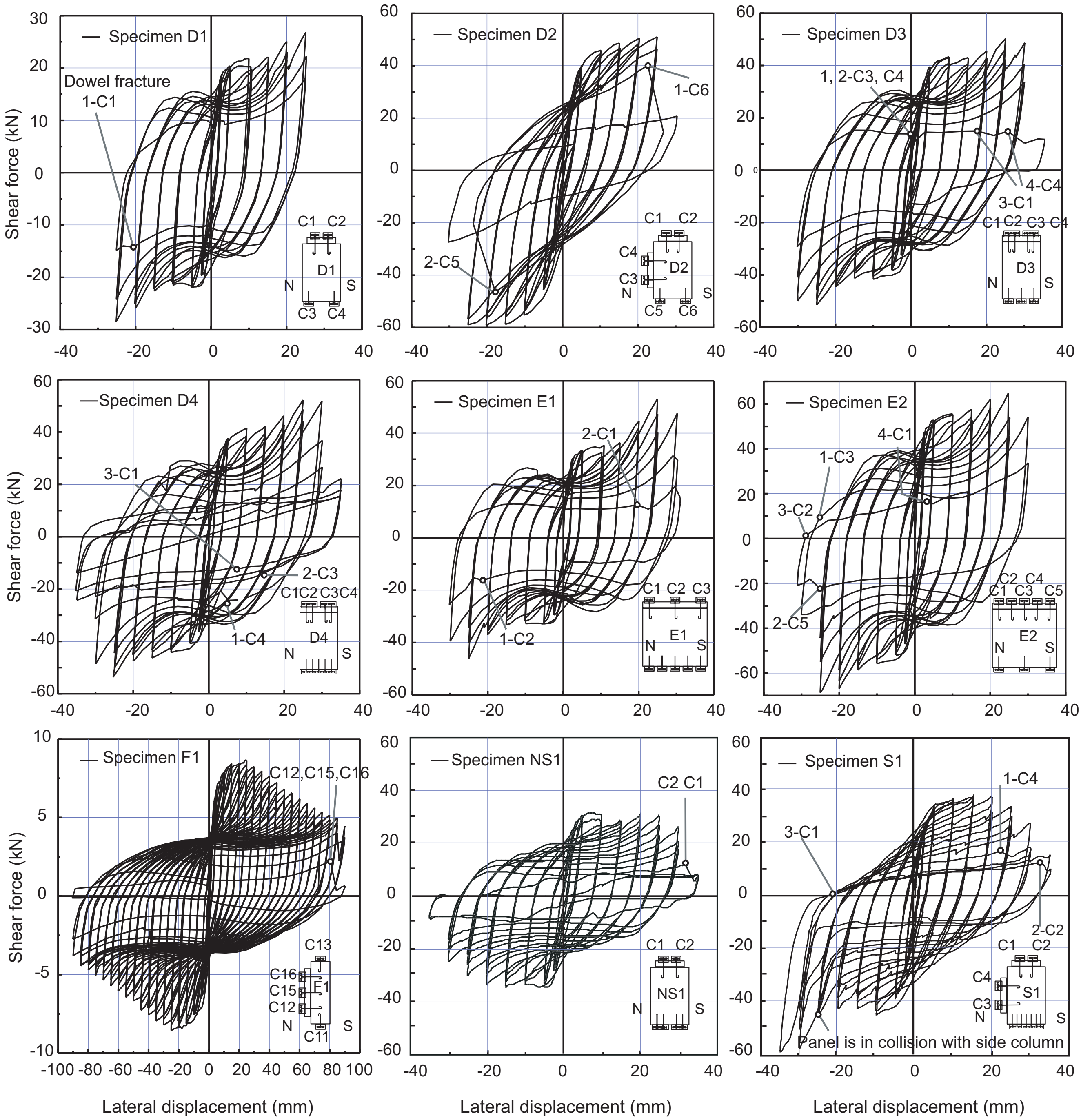

Fig. 3 Lateral load-displacement relations

Fig. 4 Failure process for connector C2 of Specimen E1

4. DISCUSSION ON TEST RESULTS

4.1 Data reduction

The envelopes of lateral load-displacement of 9 specimens are compared in Fig. 5. It is observed that all the specimens showed yielding at a story drift less than 0.25%. They all reached or maintained their maximum shear strength at story drift from almost 1.25% to 1.5%.

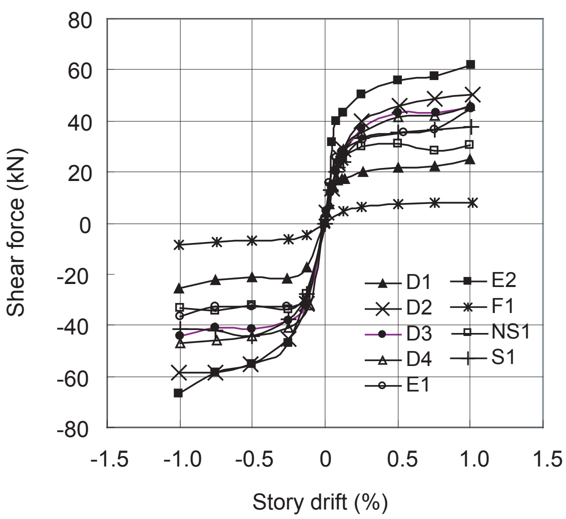

Fig. 5 Hysteresis envelopes

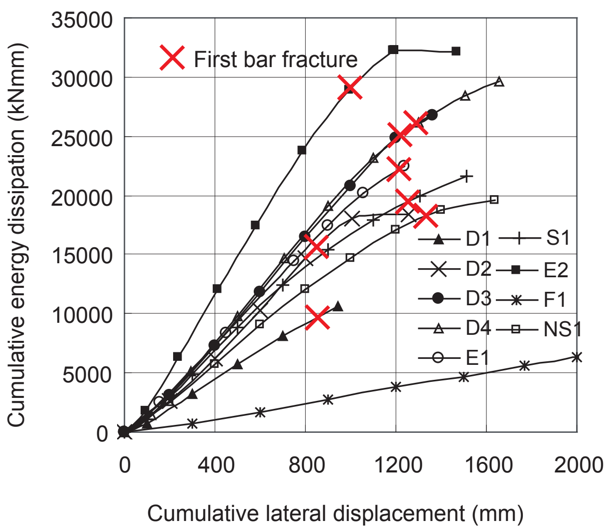

Fig. 6 Energy dissipation

Figure 6 compares the cumulative energy dissipation. The energy dissipation of Specimen F1 is small due to the gap at its bottom connector, which weakened the interaction between the panel and primary structure.

A cyclic stiffness defined in Fig. 7 was calculated. The three cyclic stiffness values obtained were averaged to get stiffness K_i. The ratios K_i/K_1 were plotted against the relative lateral displacement in Fig. 8. Stiffness degradation took place in all specimens as drift increased. It seemed that the ratio of stiffness degradation with the increasing of drift is almost same in spite of their various design parameters.

4.2 Effect of Side, Bottom, and Top Connectors

Upper connectors play a most important role in the behavior of panels. Upper connectors undertook a majority of the imposed inter-story drift, thus tended to dissipate most of the energy. From Figs. 5 and 6, it is observed that Specimen E2 had higher shear strength and better energy dissipation capacity than E1. It is due to the large amount of upper connectors for E2. Comparison of D1 with D3 can also give the same results.

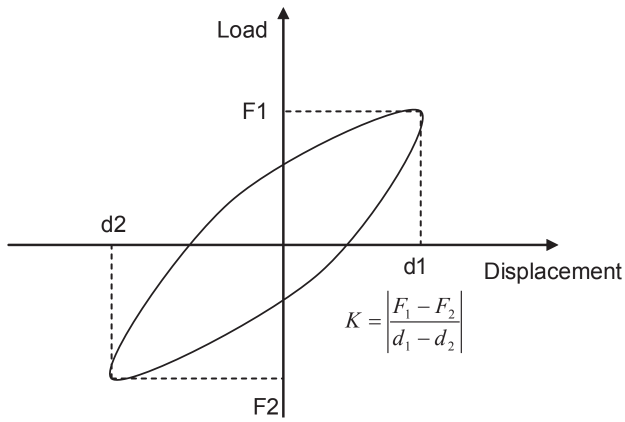

Fig. 7 Definition of cyclic stiffness

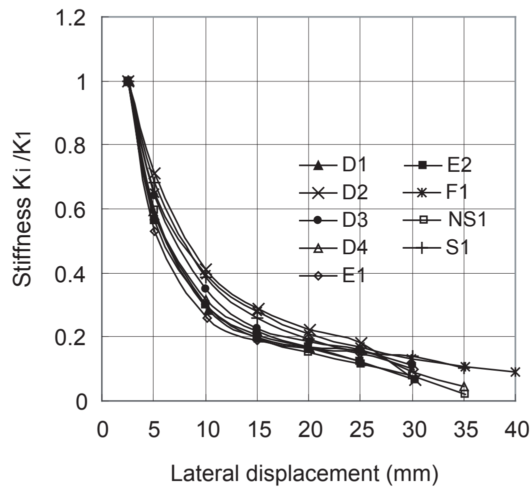

Fig. 8 Stiffness degradation in K_i/K_1

The effect of side connectors can be observed by comparing the results of specimens D1 and D2. They are same except that D2 has additional 2 side connectors. From Figures 5 and 6, it is observed that D2 had higher shear strength, almost 2 times of D1.While both reached their maximum strength at almost the same drift, D2 had better energy dissipation capacity than D1. But it is noted that D2 and D1 had different failure mode as mentioned in previous. The side connectors leaded to a large increase of the horizontal and vertical relative displacements on bottom connectors. This might give rise to the earlier failure of the dowel bars in bottom connectors. However, this failure mode can be avoided if enough steel bars being used in the bottom connectors. Compared with Specimen D2, Specimen S1 was designed with smaller diameter steel bars in 2 side connectors and 2 top connectors, but 7 steel bars in the bottom. In this situation, there is no damage as D2 being found in the bottom for S1.

A certain number of bottom connectors are needed to ensure safety in the bottom connectors as mentioned in above. But the increase of additional bottom connector amounts has no visible effect on the behavior of precast concrete partition walls. For D3 and D4, the only difference of design parameters between them is that D3 has 3 bottom connectors while D4 has 5. It can be seen from Figs. 5, 6, and 8 that the performance of D3 and D4 appeared to be almost the same.

4.3 Effect of Gap Distance in Bottom Connectors

Gap distance of 20 mm was introduced in bottom connectors of F1. Figures 5 and 6 indicate that F1 had very good deformation capacity, while its energy dissipation capacity was very poor due to the lower shear strength. Even the 3 side connectors could not help. Therefore, the introducing of gap distance in bottom connectors weakens the interaction between precast concrete partition walls and primary structure.

4.4 Effect of Spiral Reinforcement around Dowel Bars

From the failure process of Specimen NS1, it is concluded that spiral reinforcements around the dowel bars was not necessary. The failure was still caused by the fracture of the dowel bars.

5. CONCLUSIONS

It is concluded from the test results:

1. The non-structural isolated concrete partitions showed desirable deformation capacity and energy dissipation capacity under cyclic lateral loading.
2. Side connectors were effective in improving the energy dissipation capacity of the non-structural partitions. The introducing of side connectors may change failure mode. However, the unfavorable failure mode can be avoided if dowel bars are added in the appropriate location.
3. Connectors which took a majority of the imposed inter-story drift tended to dissipate most of the energy.
4. Introduce of gap at both top and bottom weakened the interaction between precast concrete partition walls and primary structure. This made the partition walls had better deformation capacity but poor energy dissipation capacity.
5. Spiral reinforcements around the dowel bars were not necessary in this test.

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REFERENCES

Architectural Institute of Japan (1999). Design guidelines for earthquake resistant reinforced concrete buildings based on inelastic displacement concept. (in Japanese)

Darama H. (2006). Seismic Performance of Non-Structural Precast Concrete Walls in RC Buildings. Ph.D thesis, Department of Architecture, The University of Tokyo, Tokyo.

Kaori OKUBO, Darama H., Hitoshi SHIOHARA, and Kazuo TAMURA (2006). Test on Seismic Performance of Non-Structural Precast Concrete Partitions Jointed by Dowel-Bar Connectors. The 12th Japan Earthquake Engineering Symposium, Tokyo, Japan. (in Japanese)

PCI Committee on Connection Details (1998). Standard Precast Connections. *PCI Journal* **43:4**, 42-58.

Syam S. Mannava, Thomas D. Bush, Jr., and Anant R. Kukreti (1999). Load-Deflection Behavior of Smooth Dowels. *ACJ Structural Journal* **96:6**, 891-898.

Vintzeleou, E. N., and Tassios, T. P. (1987). Behavior of Dowels under Cyclic Deformations. *ACJ Structural Journal* **84:1**, 18-30.

William G. Davids, and George M. Turkiyyah (1997). Development of Embedded Bending Member to Model Dowel Action. *Journal of Structural Engineering, ASCE* **123:10**, 1312-1320.