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SEISMIC PERFORMANCE OF FLAT SLAB SYSTEM WITH PRE-CAST PANELS CONNECTING TO PERIMETER COLUMN

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SUMMARY

In a seismic design of RC flat slab system, it is important to prevent its punching failure around a column. The AIJ standard for RC structures provides the equation to assess the punching strength. Recently in Japan, large scaled flat slab system is usually used in super high-rise RC buildings with high strength materials and pre-cast components. Because of lack of test data, it is necessary to verify the conventional seismic design method for such type of flat slab system experimentally. Half-scaled specimens simulating flat slab and perimeter column connections were tested under static and cyclic loading reversals to investigate their seismic performance including punching failure behavior. In the specimens, pre-stressed and pre-cast void panels with 70 MPa strength of concrete were used in the half bottom part of the slab. The test results indicated that the practical seismic design using the conventional equation for punching strength was sufficiently safe, provided that enough shear reinforcement was arranged around a column. Moreover, no wrong effect of the pre-cast panel was observed on the ductile behavior of the system. However, in case of insufficient shear reinforcement, the punching strength by the conventional equation was over estimate due to torsional failure of the perimeter beam. It was clarified from the three dimensional non-linear FEM analysis that the shear reinforcement in the perimeter beam is effective to prevent torsional failure and realize the punching strength.

INTRODUCTION

Flat slab construction is gradually used in super high-rise reinforced concrete buildings recently in Japan. This system allows flexible partition of space without beams or columns. However, in earthquakes, the transfer of bending moments between slab and column in this structure causes high stresses near the column faces. Because of the high shear stresses, this system could lead to brittle punching shear failure around slab-column connections. In the standard for reinforced concrete structures of Architectural Institute of Japan (AIJ), the punching strength is defined as summation of bending moments and shear strengths in the front and rear faces of a column and torsional strengths in the side faces of a column. In spite of practical uses, only a few research projects have been conducted on the punching shear resistance

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of concrete slabs with pre-stressed pre-cast concrete panels. This research program involved the structural test of reinforced concrete flat slab-perimeter column connections simulating a part in a prototype flat-plate building (Fig.1). In the test, specimens with different bar arrangements of slab reinforcement were subjected to gravity and lateral load. They were designed to fail by bending of the slab or to fail by punching in the surrounding area of the column. The influence of the loading direction or the amount of slab reinforcement on the failure mode, maximum strength and ductility was investigated. Moreover, three dimensional non-linear FEM analysis was carried out to simulate the behavior of specimens and to investigate the effect of the amount of shear reinforcement in the perimeter of the slab.

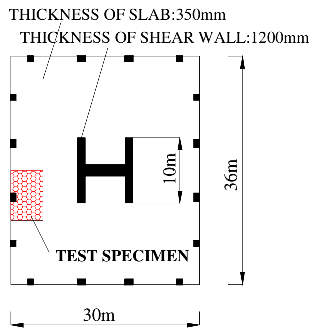

Fig. 1 Floor plan of prototype structure

OUTLINE OF EXPERIMENT

Test specimens

Three half scaled flat slab specimens simulating the perimeter part in super high-rise reinforced concrete building were tested. The specimens consisted of a pre-cast column, pre-cast void plates and cast-in-place topping concrete. Reinforcement details of specimens are shown in Fig.2 and their outlines are listed in Table 1. In all specimens, the dimension of the slab was 3600mm x 2800mm square with 175mm thickness and the section of the column was 475mm x 475mm square. Exterior of slab was reinforced like as a beam with main bars and stirrups. The beam depth was almost equal to the thickness of the slab and the width was equal to a column width. The characteristics of three specimens were as follows.

TYPE 1 : A specimen simulating practical structural design to fail by bending of slab

TYPE 2 : A specimen designed to fail by punching near the slab-column connection.

TYPE 3 : Reinforcement details were same as TYPE1 specimen. Load direction was different from TYPE1.

The punching strength of TYPE1 and TYPE3 specimens were designed to be significantly lower than the yielding strength of the slab. TYPE2 specimen was arranged with a lot of slab rebar and without exterior beam stirrups from the face of column to 300mm length to realize punching failure in the slab. Half bottom part of a flat slab was commonly composed by pre-stressed pre-cast void concrete panels. Details of the pre-cast void concrete panel is shown in Fig.3. The panel was pre-cast concrete board with void grooves between ribs. The concrete was pre-stressed by PC strand along the ribs to sustain large gravity load. Material properties of specimens are listed in Table2.

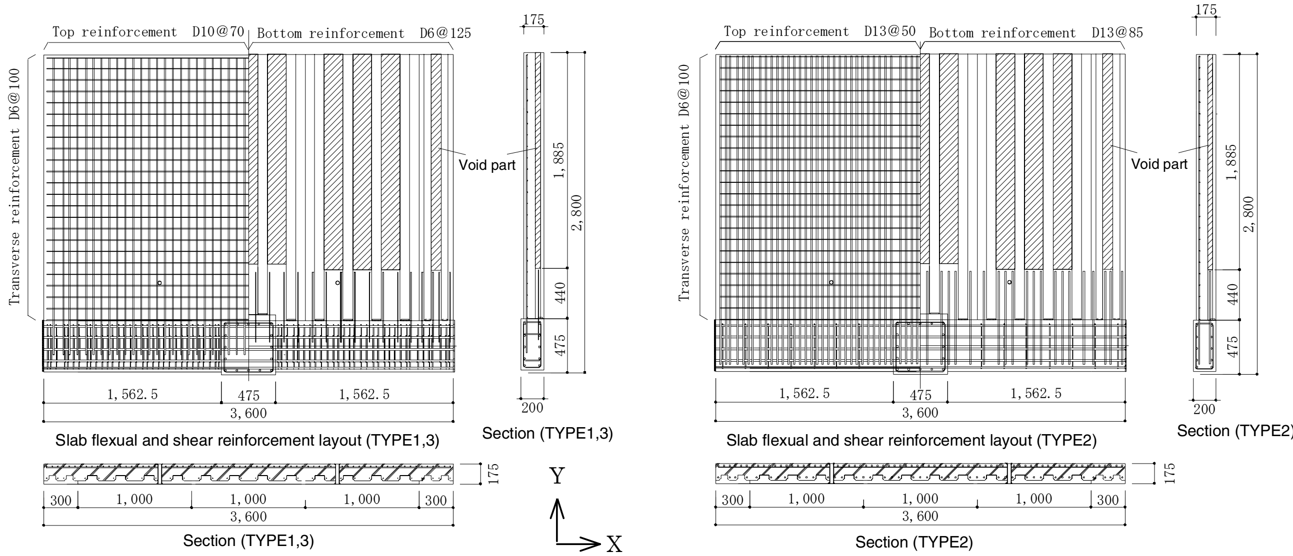

Fig. 2 Slab flexural and shear reinforcement layout

Table 1 List of test specimen

Specimen	Direction of loading	Top reinforcement	Transverse reinforcement	Bottom reinforcement	Column reinforcement	Beam reinforcement
TYPE 1	Y Direction	D10@70	D6@100	D6@125	Main 16-D16	Main 5-D16
					Hoop 2-D6@50	ST 4-D6@50
TYPE 2	Y Direction	D13@50	D6@100	D13@85	Main 16-D19	Main 5-D16
					Hoop 3-D10@50	ST 2-D6@200
TYPE 3	X Direction	D10@70	D6@100	D6@125	Main 16-D16	Main 5-D16
					Hoop 2-D6@50	ST 4-D6@50

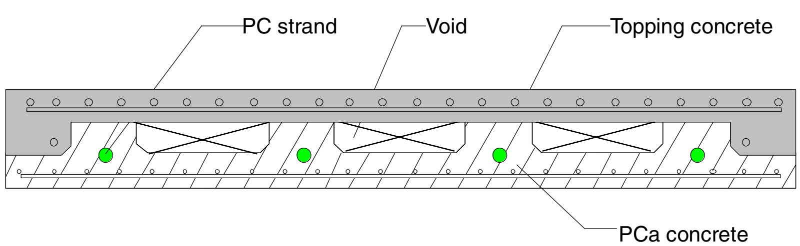

Fig. 3 Section details of slab

Table 2 Material properties

Concrete		
	Compressive strength (MPa)	Young's module (GPa)
Column	95.1	40.97
PCa plate	71.5	34.35
Topping concrete	41.75	33.63

Steel				
	Young's module (GPa)	Yield strength (MPa)	Yield strain (%)	Tensile strength (MPa)
D6 (SD295A)	175	407	0.25	526
D10 (SD295A)	185	357	0.20	496
D13 (SD295A)	183	355	0.20	532
D16 (SD390)	180	428	0.27	630
D19 (SD390)	191	438	0.27	621

Loading Method

Loading apparatus is shown in Fig.4. The column was fixed horizontally to the reaction floor so that the slab was set up vertically. A constant equivalent dead and live load (an equivalent gravity load) was loaded at two points near the column in the slab. The magnitude of the load was 31.4 kN at each point. Loading direction of each specimen to simulate an earthquake lateral load is shown in Fig.5. In specimen TYPE 1 and 2, the direction of lateral load was assumed to be Y-direction defined in Fig.2. Two corner parts of the slab were loaded reversibly by hydraulic jacks to generate same relative deflection against the fixed column. In specimen TYPE3, the direction of lateral load by earthquake was assumed to be X-direction defined in Fig.2. Four corner parts of the slab were loaded in this specimen. Control rule is illustrated in Fig.5. Jacks No.1 and No.3 were controlled to move to the same direction, and jacks No.2 and 4 were also controlled to move to the same direction but the opposite direction to No.1 and No.3. Moreover, based on the result of three dimensional structural analysis for the plototype structure, the magnitude of the deflection at loading points by No.3 and No.4 was maintained twice as large as that at loading points by No.1 and No.2. Loading history is shown in Fig.6. In specimen TYPE 3, target rotation angle was defined by the deflection at points by No.1 and No.2. Loading was controlled by rotation angles up to the final large deflection reversals with amplitude of 8.0 percent.

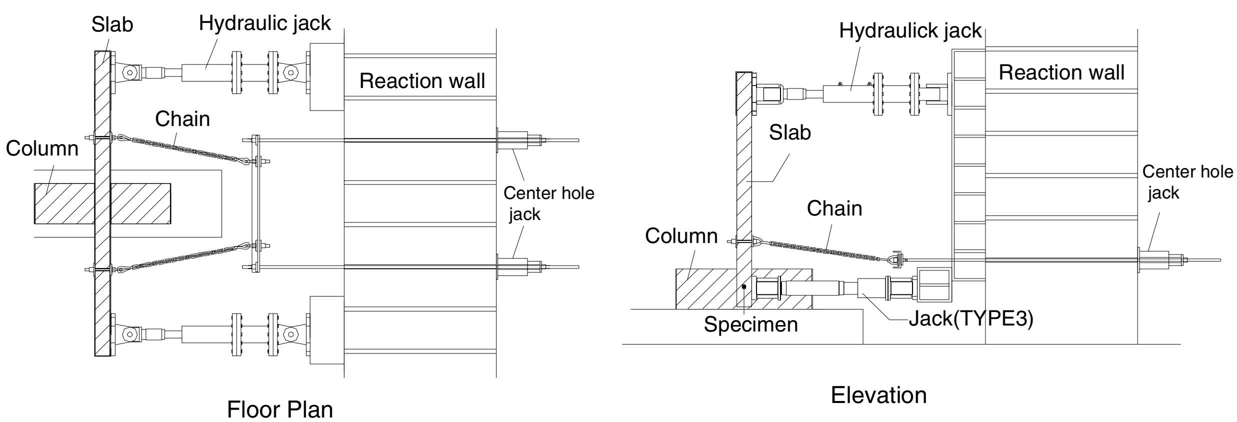

Fig. 4 Loading apparatus

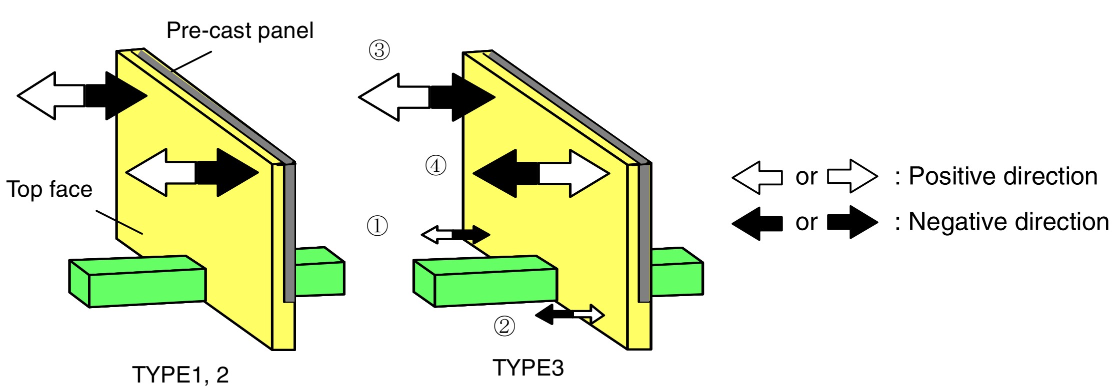

Fig. 5 Loading direction

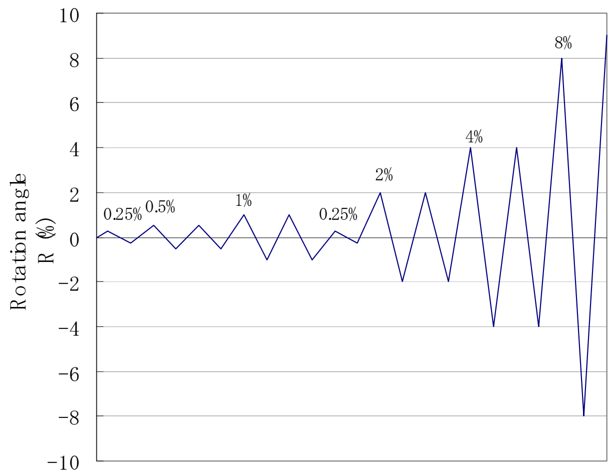

Fig.6 Lateral history

TEST RESULTS

Damage Patterns

Test results are summarized in Table 3, together with calculated strengths. Final crack patterns are illustrated in Fig.7. In specimen TYPE 1, initial flexural cracks occurred along boundary between slab and beam. Many flexural type cracks extended in the slab as the deflection became large. Especially, cracks along the boundary between pre-cast concrete panel and beam significantly opened during large positive loading cycles. Final failure mode was flexural failure in full slab. In specimen TYPE 2, cracks in the slab extended radially from the column. Characteristic cracks by torsional effect were observed in the side of the beam near the column where no shear reinforcement were arranged. Cracks were concentrated near the column. Final failure mode was punching failure in the slab near the column. In specimen TYPE 3, inclined flexural cracks generated in beams extended into the slab. Flexural behavior was dominant up to large deformation. However, final failure was a separation of pre-cast concrete panel from the cast-in-place topping concrete.

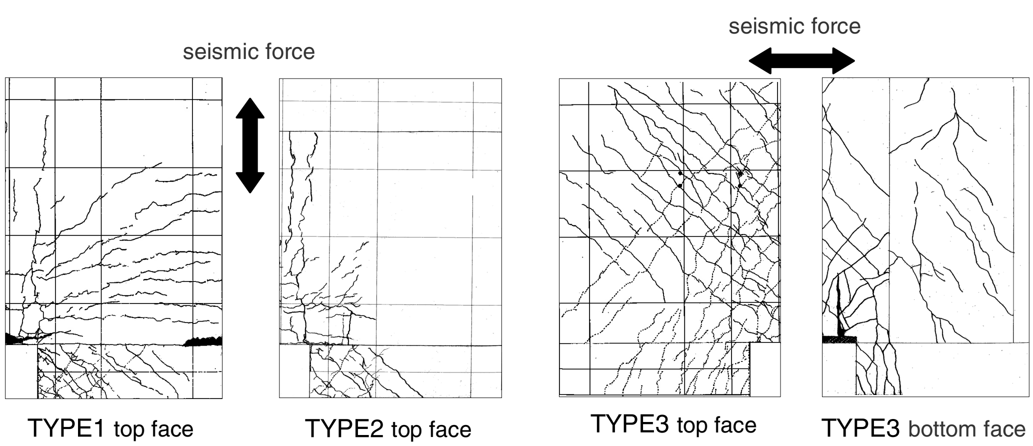

Fig. 7 Final crack patterns

Table 3 Summary of test results

Specimen		Initial stiffness (kN/mm)*1		Crack strength (kN)		Punching strength (kN・m)*2				Peak lateral load (Test result)		Failure mode
		Calculated value	Test result	Calculated value	Test result	Ms	Mt	Mf	Mo	Load(kN・m)	Rotation angle(%)	
TYPE1	+	17.84	4.06	-21.15	-20.61	63.4	138.8	10.1	212.4	86.8	4.0	Full slab flexural failure
	-							-34.2	-236.5	-176.4	-4.0	
TYPE2	+	20.05	5.61	-25.29	-20.41	63.4	138.8	50.0	252.2	122.6	2.0	Punching failure (beam torsional failure)
	-							-73.0	-275.2	-174.7	-1.8	
TYPE3	+	81.9	20.77	23.86	21.56	106.5	84.0	127.9	318.4	227.1	6.2	Separation pre-cast plate from slab
	-									-203.9	-3.9	

*1 : $K=3E_cI_e/L^3$ (Ie : geometrical moment of inertia, L : Arm length from the loading point) *2 : Punching strength capacity by AIJ standard

Load-Deflection relationship

Load-deflection relationships obtained from the test are shown in Fig.8. In the figures, calculated ultimate flexural strength and calculated punching strength of the slab are plotted by dotted lines and dashed lines, respectively. Punching strength was calculated according to the following equation by the AIJ standard.

$$\frac{V_u}{V_0} + \frac{M_u}{M_0} = 1 \tag{1}$$

where, V_u : Ultimate vertical load possible to transmit

M_u : Ultimate moment possible to transmit

V_0 : Ultimate vertical load possible to transmit, subjected to only vertical load

M_0 : Ultimate moment possible to transmit, subjected to only moment load

$$M_0 = M_f + M_s + M_t \tag{2}$$

where, M_f : Moment at the calculated front and rear sections possible to transmit by bending resistance

M_s : Moment at the calculated front and rear sections possible to transmit by shear resistance

M_t : Moment at the calculated side sections possible to transmit by torsional resistance

In specimen TYPE 1, applied load reached the maximum at 4 percent of rotation angle in both positive and negative loading directions. Up to the large displacement, a desirable energy dissipating capacity was demonstrated. The maximum strength in the positive direction coincided with the flexural strength calculated by assuming the full effective flange width and the tensile strength of bottom reinforcement of the slab. In negative loading direction, the maximum strength was well predicted in case of assuming three quarters effective flange width and the yield strength of top reinforcement in the slab. In specimen TYPE 2, the maximum strength was demonstrated at 2 percent of rotation angle in both loading directions. However, subsequently, the applied load decreased rapidly, especially in negative loading direction. The strength of this specimen was about 30 percent lower than the calculated punching strength. The observed torsional shear failure in the beams was estimated to affect strongly the lower strength of this specimen. In other words, the strength of the specimen depended on the amount of shear reinforcement in the beam. In specimen TYPE 3, observed strength well coincided with the calculated flexural strength. No significant deterioration in strength was observed up to the final loading reversals.

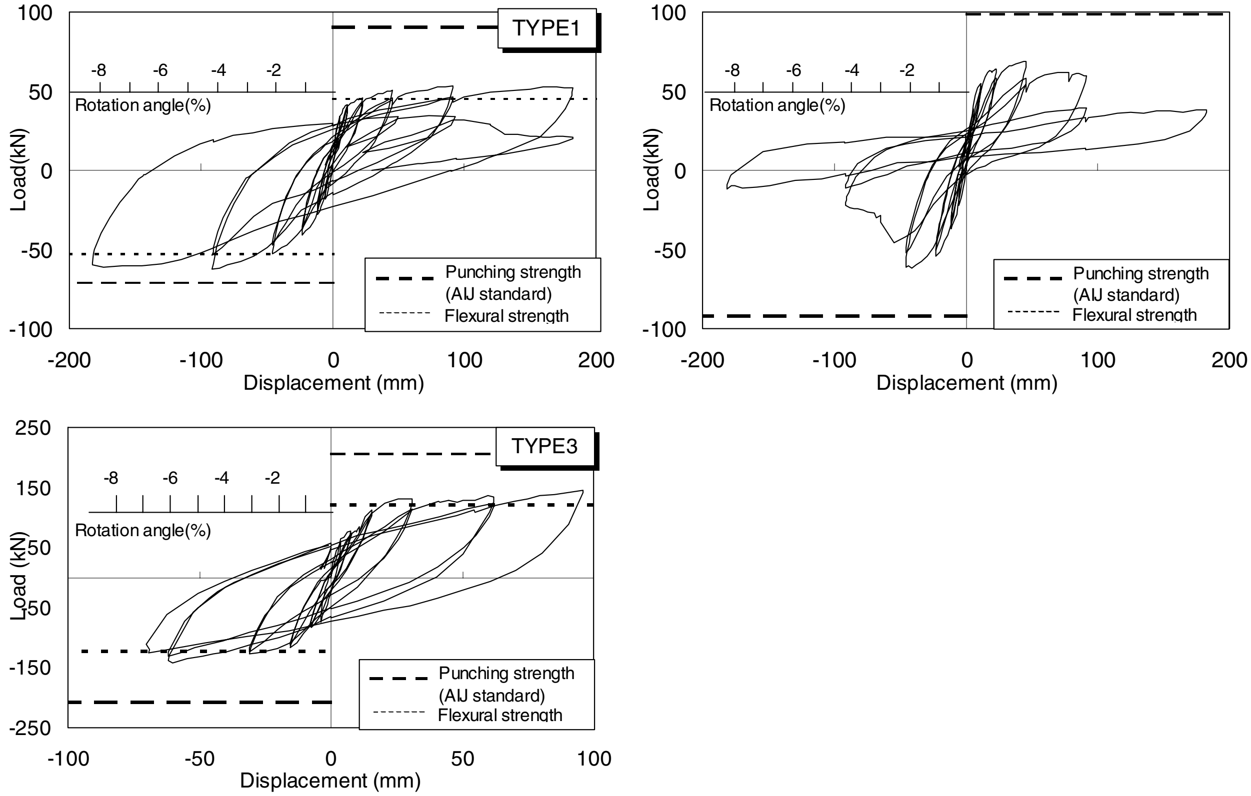

Fig. 8 Load-Displacement relationship

FEM ANALYSIS

Analysis method

A three-dimensional nonlinear finite-element method (FEM) was carried out to simulate the behavior of specimens and to study the effect of the amount of shear reinforcement in the beam. A general-purpose computer program, MARC, was used for the analysis. The finite element meshes for the flat slab specimens were shown in Fig.7. Each half part of the tested specimens TYPE 1 and TYPE 2 was analyzed, considering symmetric property in their shape and loading conditions. Boundary condition was same as the experiment. The concrete element was modeled by 8-Nodes solid block, and the reinforcing steel was modeled as 2-Node truss element.

The uni-axial stress-strain relation for concrete in compression and tension was assumed as shown in Fig. 8(a). Concrete modeling in compression was based on Von Mises yield surfaces and was defined as isotropic material. Poisson's ratio was assumed to be 0.2. Under compressive stresses, the curve was simulated as quadric curve until reached strain ε_{cu} =0.002. After that, the stress was assumed to decrease till a point (4 ε_{cu}, 0.2f_c) and then to maintain the constant level. Concrete under tensile stresses was modeled as low-tension material, considering cracking behavior. Immediately after the maximum principal stress exceeded the tensile strength, cracks and linear tension softening were assumed.
This low-tension material model was employed in the modeling of cast-in-place topping concrete. However, the pre-cast concrete was modeled as an elastic material both in compression and tension, because the pre-cast concrete plates were pre-stressed using high strength concrete and no crack was observed in the test. The reinforcing steel was modeled as bi-linear relationship in consideration of the

strain hardening (Fig.8 (b)). Bond slip between concrete and reinforcement was not considered in the analysis.

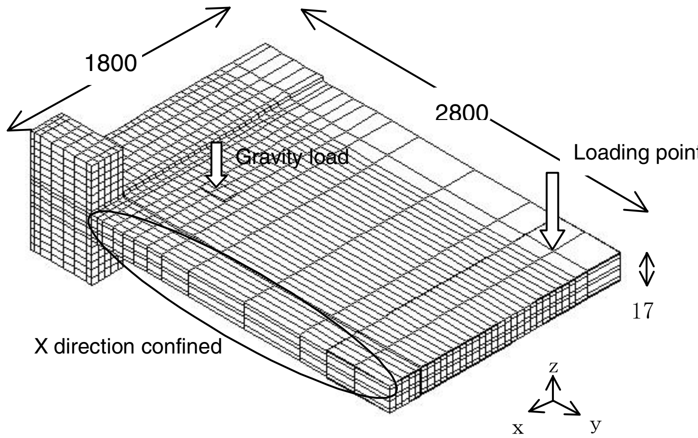

Fig. 9 Finite Element meshes (dimensions in mm)

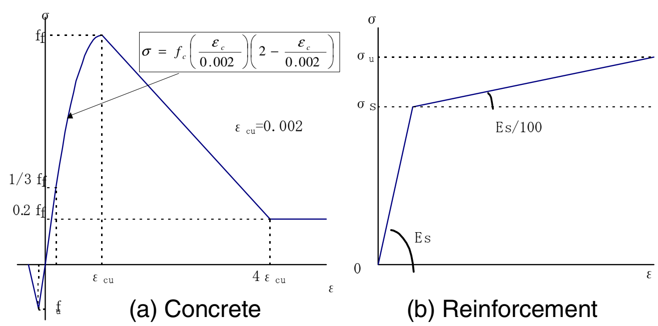

Fig. 10 Assumed stress-strain model of materials

Comparison with test results

Analyzed cracks development in specimen TYPE 1 was almost similar to the observed test results. The distribution of the equivalent tensile strain near the column in the analyzed specimen TYPE 2, which is equivalent to cracks extension, is illustrated in comparison with photograph during the test in Fig.11. In the figure, a part of deep color represented the area of large tensile strain. Large strain area appeared in an oblique direction on the side of the beam. Obviously, torsional behavior was dominant also in the analysis.

Load-displacement relationships obtained from the analysis are compared with the test results in Fig.12. Analysis reproduced non-linear hysteretic behavior up to the maximum strength with tremendous accuracy in both specimens TYPE 1 and TYPE 2 in spite of different failure modes.

Analyzed strain distributions in top reinforcement of a slab at the section with the maximum moment are compared with test results in Fig. 13. The phenomenon that yielding started near the column and then extended gradually into whole slab was reproduced by the analysis in the flexural type failure.

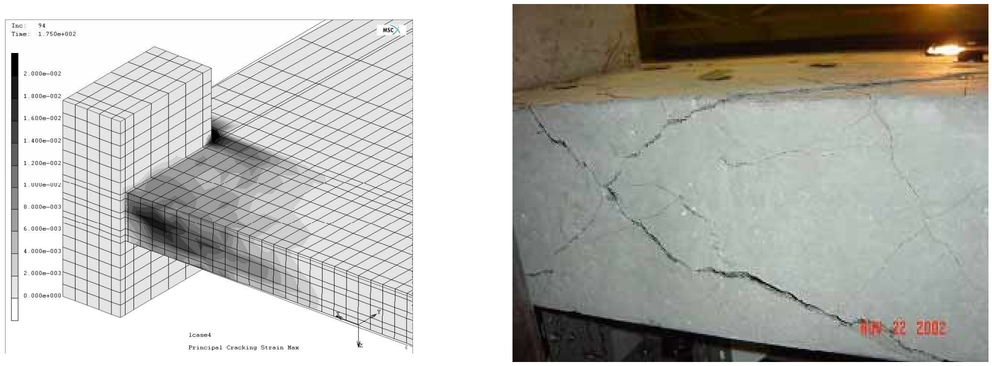

Fig. 11 Crack condition

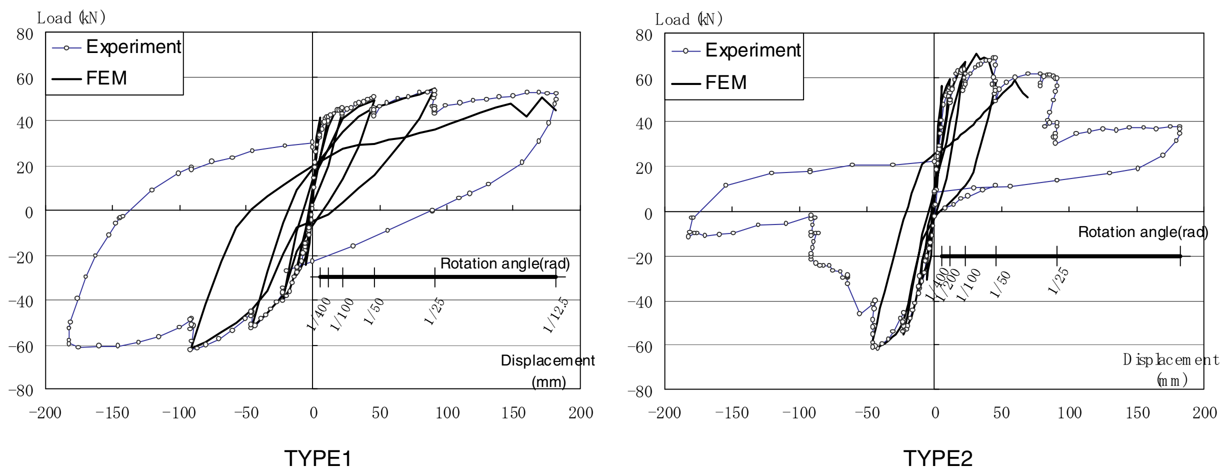

Fig. 12 Load-displacement relationship

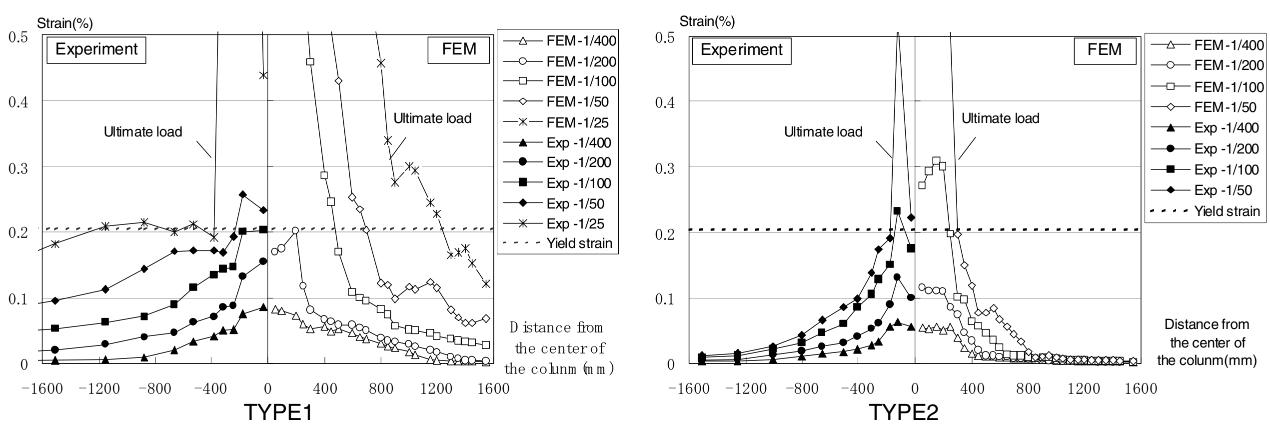

Fig. 13 Strain distribution

Effect of shear reinforcement

Good compatibility between analysis and test results was verified. Therefore, the effect of the amount of shear reinforcement in the perimeter beam on the ultimate strength caused by the failure near the column was investigated by the same analytical method as above mentioned. Parameters in the analysis was only the amount of the shear reinforcement in the analytical model of specimen TYPE 2. Six cases were analyzed in the same manner as the analysis to simulate the test.

Obtained relationship between the ultimate strength and the ratio of shear reinforcement of the beam was plotted in Fig. 14. Dashed line in the figure represents the calculated punching shear strength according to the AIJ standard. The ultimate strength significantly depended on the amount of the shear reinforcement. The smaller ratio of the shear reinforcement represented the lower ultimate strength. In case of 0.14 percent of shear reinforcement ratio, dominant torsional cracks in the beam were conformed, that means the lower ultimate strength was caused by the torsional failure in the beam. In cases of the ratio larger than about 0.6 percent, ultimate strengths equivalent to the calculated punching strength was developed. It is concluded that the shear reinforcement in the perimeter beam is effective to prevent torsional failure and realize the punching strength.

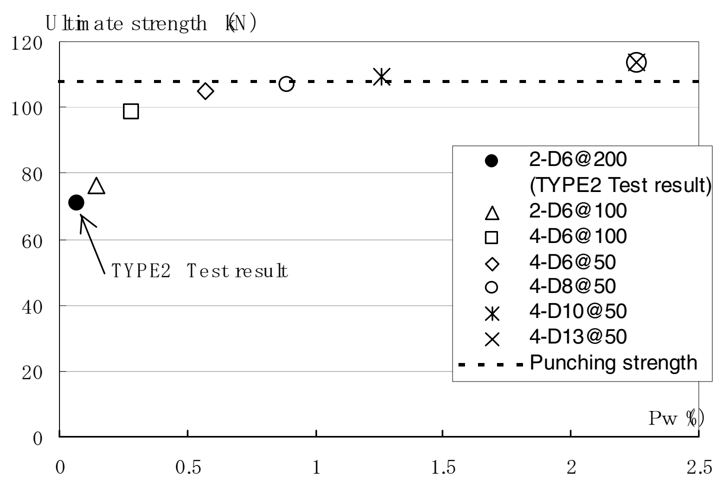

Fig. 14 Ratio of beam shear reinforcement-ultimate strength relationship

CONCLUSIONS

Three perimeter flat slab-column connections were tested under gravity and cyclic lateral loading. The influence of loading direction and bar arrangements of slab reinforcement was investigated. Based on the test results and FEM analyses, the following conclusions were obtained.

1) Specimens based on the practical design demonstrated desirable flexural behaviors and large energy dissipating performance regardless of loading directions. No wrong effect of the pre-stressed pre-cast concrete panels used in the slab was observed on the ductile behavior of the structural system. The ultimate strength was predictable based on the conventional flexural theory.
2) Three dimensional FEM analysis was able to reproduce the non-linear behavior of the structural system in both cases of flexural and punching failure modes.
3) In case of insufficient shear reinforcement in the perimeter beam, the punching strength predicted by the conventional equation was over estimate, because of torsional failure in the beam. It was clarified that

the shear reinforcement in the perimeter beam is effective to prevent torsional failure and realize the punching strength.

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