STRUCTURAL PERFORMANCES OF PRESTRESSED CONCRETE INTERIOR BEAM-COLUMN JOINTS

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SUMMARY

Four one-third scaled PC interior beam-column joints were tested by the authors. The main parameters of this test were the prestressing force and the bond condition between grouted mortar and prestressing tendons through a joint. The compressive strength of concrete and grouted mortar was 36 MPa and 40 MPa, respectively. The yield strength of prestressing tendon was 1100 MPa. Columns and beams of the specimens were reinforced by deformed bars with the yield strength of 500 MPa. Prestressing force was given from the ends of beams using post-tension method. These beam-column joints were analyzed using nonlinear finite element method (FEM). This analysis was carried out by considering the bond between grouted mortar and prestressing tendons similar to the tests.

All specimens failed in the joint shear failure with both test and FEM analysis. The FEM analytical results gave a good agreement with the test results on the maximum story shear forces and the failure mode. From the comparisons between the experimental and FEM analytical results, the effects of the prestressing force and the bond condition between grouted mortar and prestressing tendons on the joint shear capacity were discussed.

From the detailed investigations, the following conclusions can be made:

(1) The joint shear cracking strength obtained from the tests was almost constant. However, the angles of shear cracking in the joint changed with the prestressing forces.

(2) The shear capacity of the joint was not especially affected by the prestressing forces.

(3) Because the bond between grouted mortar and rounded tendons was vanished on the earlier loading stage, the effect of the bond conditions on the joint shear capacity was not obtained obviously.

INTRODUCTION

In the Cooperative Research, “Research and Development of Structural Design and Construction Guideline for Prestressed Concrete Structures” organized by Japan Association for Building Research Promotion, a number of research for establishing the rational design method of prestressed concrete (PC) buildings has been carried out since 1995. This study has been conducted as a part of the work in the Working Group for Evaluation of Structural Performance (Head: Professor F. Watanabe, Kyoto University) under Research Coordinating Committee (Chairman: Dr. S. Okamoto, former Director General of Building Technology Research Institute in Building Center of Japan). In order to establish the rational design method of PC beam-column joints under seismic forces, both the experimental and FEM analytical studies have been carried out.

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Table 1: Properties of specimens

Specimen		PC-0	PC-1	PC-2	PC-U
Column	Main bars Hoop	12-D19(SD490) 4-D10@40(SD785), pw =1.2%			
Joint	Hoop	2-D10 x 1set, pw = 0.22%			
Beam	Main bars Stirrups Tendons	Top bars: 3-D19(SD490), Bottom bars: 3-D19(SD490) 2-D10@50(SD785), pw = 1.43% 2-ϕ23(Grade-C, No.1, SBPR110/125)			
	Pe, N	0	216000	432000	216000
	Pe/(Ab□Fc)	0	0.1	0.2	0.1
	Grouting	○	○	○	×

Note) pw: Lateral reinforcement ratio, Pe: Effective prestressing force,
Ab: Sectional area of beam(= 60000 mm^2), Fc: Concrete strength, MPa

Table 2: Material Properties

a) Concrete (units in MPa and μ)

Concrete	Specimen	Comp. strength	Secant stiffness	Split strength
Fc36	PC-0, PC-1	34.4	28000	3.01
	PC-2	35.3	28100	3.01
	PC-U	36.9	27300	3.01

b) Reinforcement and tendon (units in MPa and μ)

Bar size	Yield stress	Yield strain	Young modulus	Max. stress
D19(SD490)	517	2850	181000	692
D10(SD785)	897	4330	207000	1070
ϕ23(SBPR110/125)	1100	5500	200000	1250

OUTLINE OF TEST

Design of Tests

It was indicated that the shear performances of PC interior beam-column joints were improved by prestressing forces in previous test results [Watanabe, 1994]. However a flexural compressive failure of beams was occurred in these specimens, the joint shear capacity was not obviously understood. As an unbonded tendon system changed the concentration of deformation at the beam critical sections, it was considered that the different shear transfer mechanisms of the joint can be formed compared with the bonded one. From these problems, prestressing forces and bond conditions of tendons were selected as the main parameters in this study.

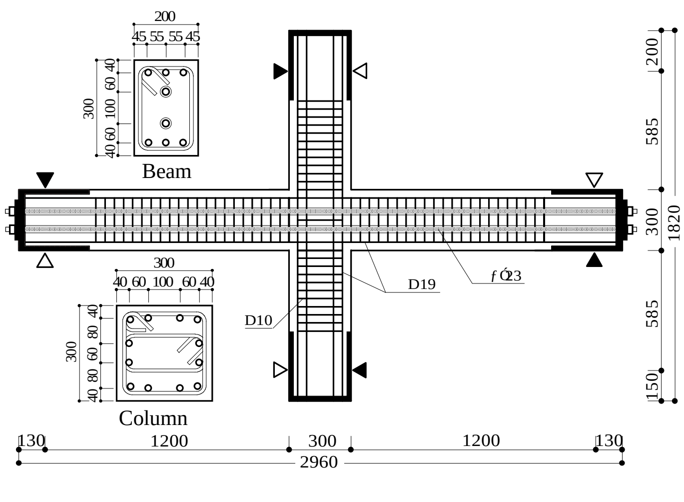

Fig. 1: Details of Specimen PC-0

Table 3: Tests results

a) At the joint shear cracking (units in kN, mm and MPa)

Specimen	Story shear force	Story displacement	${}_{exp}\tau_{crack}$	${}_{cal}\tau_{crack}$	${}_{exp}\tau_{crack}$ / ${}_{cal}\tau_{crack}$
PC-0	53.1	4.8	3.7	4.4	0.84
PC-1	52.1	3.1	3.6	5.9	0.61
PC-2	56.5	2.9	3.9	7.2	0.54
PC-U	54.0	3.8	3.8	6.0	0.63

Note) ${}_{exp}\tau_{crack}$: experimental joint shear stress, ${}_{cal}\tau_{crack}$: calculated joint shear stress

b) At the maximum story shear force (units in kN, mm and MPa)

Specimen	Story shear force	Story displacement.	${}_{exp}\tau_{max}$	${}_{cal}\tau_{max}$	${}_{exp}\tau_{max}$ / ${}_{cal}\tau_{max}$
PC-0	157	43.8	10.9	10.3	1.06
PC-1	157	44.7	10.9	10.3	1.06
PC-2	159	44.7	11.0	10.6	1.04
PC-U	147	29.4	10.2	11.1	0.92

Note) ${}_{exp}\tau_{max}$: experimental joint shear stress, ${}_{cal}\tau_{max}$: calculated joint shear stress

Specimens and Material Properties

Properties of specimens and materials were showed in Tables 1 and 2, respectively. Four one-third scaled interior beam-column joints were tested. These specimens were post-tensioned from the both beam-ends after casting of concrete. All specimens had the interstory height of 147 cm and the beam span of 270 cm, shown in Fig. 1. The dimensions of the beam and column were 200 x 300 mm and 300 x 300 mm, respectively. The beams and the column were strongly reinforced in order to prevent the flexural yielding before joint shear failure. The prestressing level was defined as a ratio of effective prestressing stress to concrete compressive strength. In this test, the prestressing level was set from 0 to 0.2 as shown in Table 1. Then effective prestressing force was assumed to be reduced to 0.95 of initial one. Under a constant axial column force of 320 kN, reversed cyclic loading was applied to the both beam-ends similar to the seismic action. Tests were continued up to a final story drift angle of 1/20 rad.. Main measuring items were story drift angle, flexural deformation of the beams and the column, joint shear distortion, strain in reinforcement and tendons and concrete compressive strain in the joint.

Tests Results

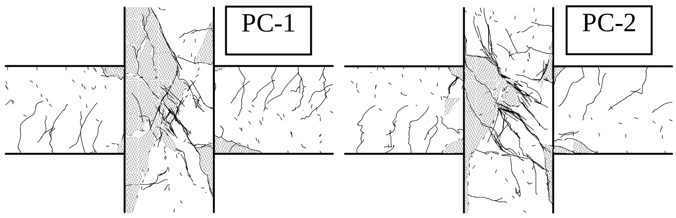

Fig. 2: Final crack patterns

Final crack patterns of Specimens PC-1 and PC-2 are shown in Fig. 2. The opening of joint shear cracks and splitting cracks along column longitudinal bars from the end of joint shear cracks were remarkable in all specimens. The angles of joint shear cracks decreased according to the increase of prestressing forces. As beam longitudinal bars and prestressing tendons of all the specimens were not yielded in the tests, it was considered that the joint shear failure occurred in all the specimens. Story shear force-story displacement relations are shown in Fig. 3. Because the structural behavior of beam-column joints at early loading stage was affected by beam flexural characters, the initial stiffness of high-leveled prestressed specimen was more rigid than a low-leveled prestressed one. It was recognized that the restoring force characteristics of all the specimens showed the reversed S-shaped type with small energy dissipation after story displacement of 23 mm, story drift angle of 1/66 rad.. Table 3 showed the test results. In this figure, experimental joint shear stresses were calculated using the definition proposed by Architectural Institute of Japan, AIJ [Architectural Institute of Japan, 1990]. Specimens PC-0, PC-1 and PC-2 reached to the maximum story shear force at the story displacement of 45 mm, story drift angle of 1/33 rad.. On the other hand, the maximum story shear force of unbonded specimen, PC-U was observed at the story displacement of 30 mm, story drift angle of 1/50 rad.. In all specimens, deterioration of shear capacity after the maximum story shear force was small. As for joint shear cracking strength, comparisons between test results and calculated value were shown in Table 3. Calculated joint shear cracking strength was derived from the modified principal stress equation (1) as follows. Prestressing effects in a joint was considered in Equation (1).

$$ {}_{cal}\tau_{crack} = (F_t^2 + (\sigma_0 + \sigma_g') F_t + \sigma_0 \cdot \sigma_g')^{0.5} \qquad (1) $$

F_t : concrete splitting strength ($= 0.506 \cdot (\sigma_B^{0.5})$), MPa
σ_B : concrete strength, MPa
σ_0 : column axial stress, MPa
σ_g' : joint prestressing stress($= (b_b/b_c) \cdot \sigma_g$, MPa
σ_g : effective prestressing stress, MPa
b_b : beam width, cm
b_c : column width, cm

Experimental joint shear cracking strength with high prestressed specimens was not accurately predicted using the Equation (1). The calculated strength overestimated the tests results over 1.5 times larger. The joint shear cracking strength obtained from the tests was almost constant. However, because the angles of shear cracking in the joint changed with the prestressing forces, it was considered that the shear cracks were affected by the joint prestressing stresses.

As for evaluation of the maximum story shear forces, comparisons between experimental and calculated shear forces were shown in Table 3. The calculated shear force was derived from the following Equation (2) proposed by AlJ [Architectural Institute of Japan, 1990].

$$ {}_{cal}\tau_{max} = 0.3\ \sigma_B \qquad (2) $$

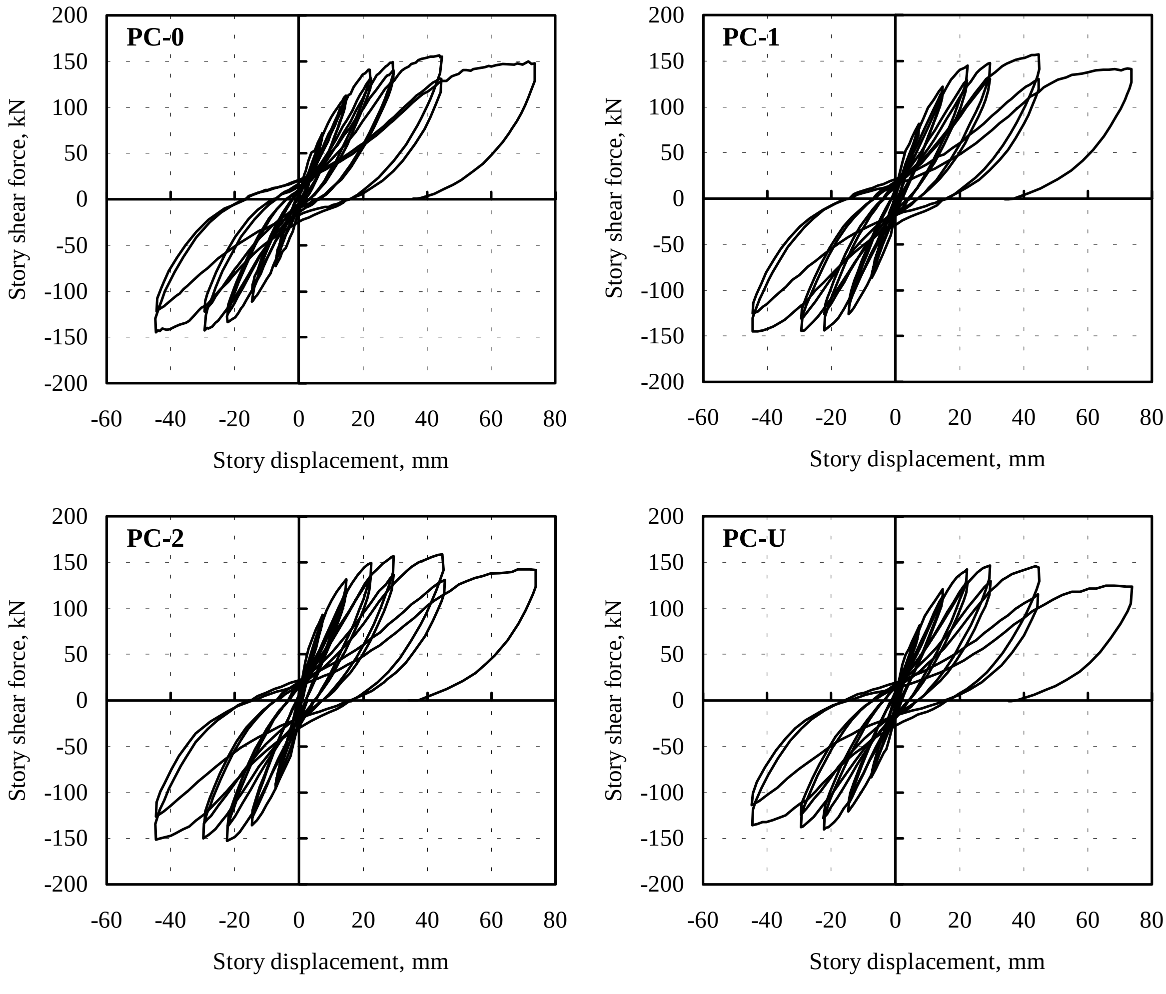

Fig. 3: Story shear force-story displacement relations

The calculated maximum story shear stress using Equation (2) gave a good agreement with the test results. The ratios of experimental and calculated maximum story shear stress, ${}_{exp}\tau_{max}$ / ${}_{cal}\tau_{max}$ were located within a range from 0.92 to 1.06. Because the experimental maximum shear forces of Specimens PC-0, PC-1 and PC-2 were not different, it was considered that the maximum story shear force was not affected by the prestressing forces at all.

OUTLINE OF FEM ANALYSIS

Reference Specimens

Four PC interior beam-column joints, PC-0, PC-1, PC-2 and PC-U were selected as the references for verification of FEM analytical program [Kashiwazaki and Noguchi, 1997]. Moreover the parametric FEM analysis was carried out to investigate the effects of prestressing force on the joint shear capacity. Prestressing forces were varied from 550 to 730 kN. Three virtual specimens, PC60, PC70 and PC80 were subjected to the effective prestressing stresses with 60, 70 and 80 % of yield stress of prestressing tendons, respectively. Specimens PC60, PC70 and PC80 were assumed to be the same shape and the same material properties with Specimen PC-0.

Analytical Method and Material Models

This FEM analysis was carried out by using the two-dimensional nonlinear FEM program [Kashiwazaki and Noguchi, 1997]. The specimens of plane interior joints were idealized using the symmetrical condition around a center point, as shown in Fig. 4. In the analysis, the incremental displacements were given to the beam-end of the specimen under constant axial loads. Boundary conditions in the analysis were applied according to the experiment. Outline of material models was described as follows.

Concrete

Six-node triangular or eight-node quadrangular element was assumed for concrete under in-plane stress condition. In this model, the element stiffness was evaluated at the gaussian points. Under the biaxial stress condition in concrete, an orthotropic hypoelastic model based on the equivalent uniaxial strain concept by Darwin and Pecnold [Darwin and Pecnold, 1977] was used for constitutive laws, and the failure was judged by the failure criterion proposed by Kupfer and Gerstle [Kupfer and Gerstle, 1973]. Ascending compressive stress-strain relationships of normal strength concrete was represented by Saenz-model [Saenz, 1964]. Confined concrete properties on the descending compressive stress-strain relationships were represented by Kent-Park's model [Kent and Park, 1971]. The reduction factor of the compressive strength of cracked concrete proposed by Ihzuka and Noguchi [Ihzuka and Noguchi, 1992] was used. Analytical concrete model was shown in Fig. 5.

Reinforcement

The main longitudinal reinforcement in columns and beams was assumed to be a linear element. The intermediate longitudinal reinforcement in columns, and the lateral reinforcement in columns and beams were

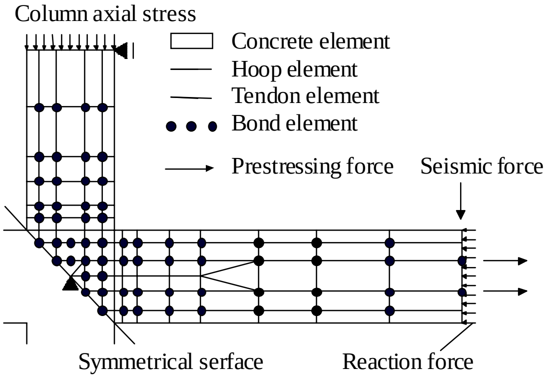

Fig. 4: Finite element idealization

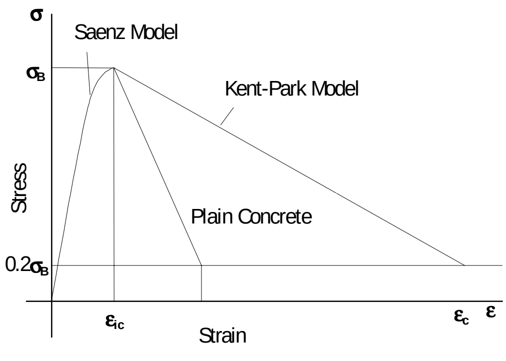

Fig. 5: Concrete model

represented as steel layers in RC elements. The stress-strain relationships of the longitudinal and lateral reinforcement were assumed to be bilinear and trilinear, respectively.

Bond and crack

Bond between longitudinal reinforcement and concrete was expressed by a discrete bond link element which is composed of two orthogonal springs. In steel layered RC elements, bond action was indirectly included using tension stiffening models. Cracks in concrete element were represented by a smeared crack model.

Prestressing force

Experimental prestressing force was introduced from the end of beam using a post-tensioning method. The analytical model for prestressing was developed to simulate the experimental prestressing procedure. Effective prestressing force and grouting was included using discrete bond link elements in this model. Thus, tendons were prestressed at first with no bond conditions, and compressive reaction force of prestressing was applied to the cross section at the beam-end. Finally the bond between prestressing tendons and grouted mortar was set using the discrete bond elements.

Test results were used for the mechanical properties of concrete, reinforcing bars and tendons. Mechanical properties of discrete bond elements were reproduced from the experiment.

Analytical Results

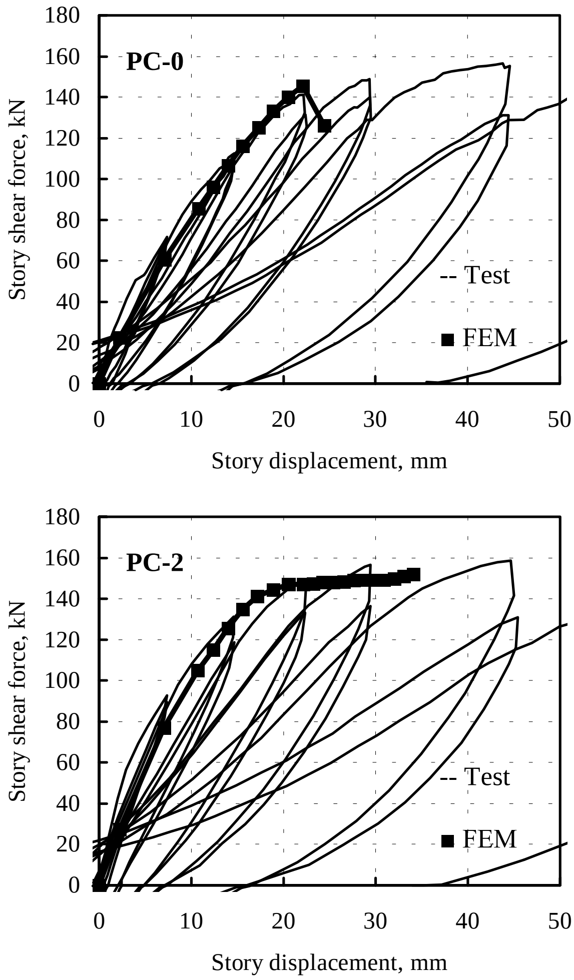

Fig. 6: Story shear force-story displacement relations

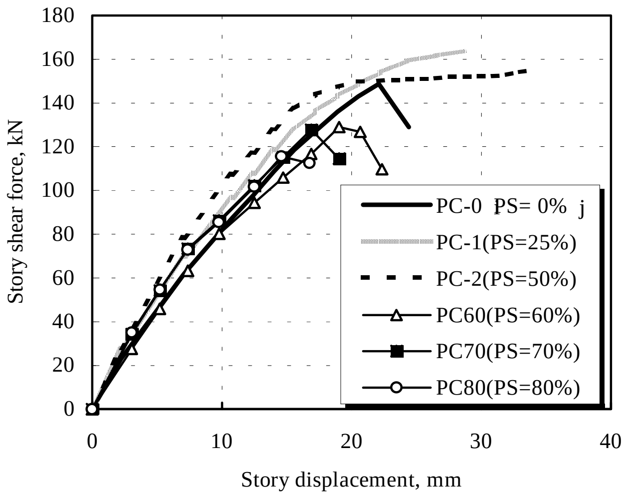

Fig. 7: Analytical results

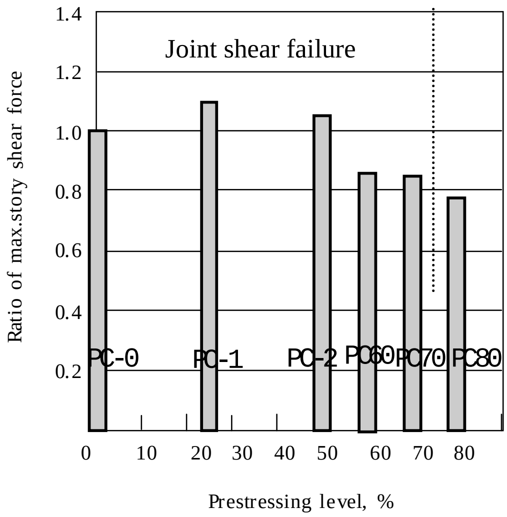

Fig. 8: Effects of prestressing levels

The analytical story shear force-story displacement relations of four specimens were shown in Fig. 6, as compared with the test results. Except for Specimen PC-U, initial stiffness of Specimens PC-0, PC-1 and PC-2 was slightly underestimated by the analysis. Results of FEM analysis gave a good agreement with the test results as for maximum story shear force. Both of the experimental and the analytical maximum story shear forces were almost constant in spite of the differences of the prestressing force within the range from 145 to 160 kN. Thus the effect of the prestressing force on the maximum story shear force was not obviously recognized in the analysis similar to tests. In all specimens, story displacement of the maximum story shear force in analysis was smaller than the test results.

Analytical results including the virtual Specimens PC60, PC70 and PC80 were shown in Fig. 7. The maximum story shear forces of Specimen PC60, PC70 and PC80 were 129, 128 and 115 kN, respectively. In analyses of Specimens PC60 and PC70, the stress condition of a few joint concrete elements reached strain softening after the peak stress at the maximum story shear force. It was recognized that the joint shear failure occurred without the beam flexural yielding. On the other hand, the beam flexural compressive failure occurred in Specimen PC80 with higher prestressing force.

The effect of the prestressing force on joint shear capacity was shown in Fig. 8. In this figure, the ratio of maximum story shear force was derived from the maximum story shear force divided by the one of Specimen PC-0 with no prestressing force. According to the increase of prestressing force, the ratios of story shear forces were located the range from 0.85 to 1.1 except for Specimen PC80 with beam flexural failure. It was recognized that the joint shear capacity was not affected obviously by the prestressing force.

CONCLUSIONS

In order to investigate the effect of the prestressing force, and the bond condition between tendons and grouted mortar on the shear capacity of PC interior beam-column joints, both the experimental and FEM analytical studies have been carried out. FEM analytical results gave a good agreement with the test results on the maximum story shear force and the failure mode.

From the discussion of experimental and FEM analytical results, the following conclusions can be made:

(1) The joint shear cracking strength obtained from the tests was almost constant. However, because the angles of shear cracking in the joint changed with the prestressing forces, it was considered that the shear cracks were affected by the joint prestressing stresses.
(2) The shear capacity of the joint was not especially affected by the prestressing forces.
(3) Because the bond between grouted mortar and rounded tendons was vanished on the earlier loading stage, the effect of the bond conditions on the joint shear capacity was not obtained obviously.

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