Experimental Study on Prefabricated Beam-Column Subassemblages

by K. Yoshioka and M. Sekine

Synopsis: Two types of prefabricated beam-column joints are devised in aiming to save manpower requirements in construction work.

The first type consists of making precast subassemblages with beam-column joints and beams integrated. Through holes are provided in the vertical direction in the beam-column joint to accommodate column reinforcing bars (Type 1).

The second type consists of making precast subassemblages with beam-column joints and columns integrated. Through holes are provided in the horizontal direction in the beam-column joint to accommodate beam reinforcing bars (Type 2).

Column reinforcing bars or beam reinforcing bars are passed through the holes in these precast subassemblages; the parts are integrated by subsequent grouting of the holes with high-strength mortar. The earthquake resistance of these precast subassemblages was investigated with cyclic loading tests. The systems are intended for use in a 13-story reinforced concrete building designed so that its collapse mechanism would be of beam-yielding type.

With precast subassemblages of Type 1, column reinforcing bars grouted and fixed inside sleeve-pipe holes are not subject to stresses extending into the plastic range. Therefore by suitably designing the anchorage lengths of beam reinforcing bars inside the joints, there will be no slippage of the beam bars. A ductility of more than 6 times the yielding displacement may be attained. With Type 2 subassemblages, the beam reinforcing bars grouted and fixed inside sleeve-pipe holes are subjected to repeated stresses extending into the plastic range such that bond deterioration occurs inside the joints. Strength declines at large deformations exceeding three times yield displacement and satisfactory ductility is not obtained. Taking test results into consideration, precast subassemblages of the first type are recommended for adoption in the 13-story building.

Keywords: Anchorage (structural); beams (supports); bond (concrete to reinforcement); bond stress; columns (supports); deterioration; failure; grouting; joints (junctions); precast concrete; shear properties; slippage

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INTRODUCTION

Precast concrete members are used in combination with cast-in-place concrete to simplify operations at the construction site and shorten the time schedule of work. Precast concrete members are also useful for alleviating the shortage of skilled labor at the site. Beam-column joints have intricated and complex arrangements of beam and column reinforcing bars. As a consequence, placing reinforcing bars in the field is difficult. Placing of concrete must also be done carefully to eliminate voids. Such problems in the field can be avoided by using prefabricated beam-column joints.

Two structuring methods described in Appendix A were devised to apply precast concrete construction to actual buildings (1,2,4).

The first type is a structuring method in which the joint panel and beams are made into a precast concrete subassemblage with sleeve-pipe holes in the vertical direction inside the joint panel. Column reinforcing bars protruding from a column are passed through these holes as the precast subassemblage is installed. High-strength mortar-base grouting material is pumped into the boundary plane between columns and joint and into the sleeve pipes to fix the precast subassemblage.

The second type is a structuring method in which the joint and columns are made into a precast subassemblage with sleeve-pipes holes in the horizontal direction inside the joint. Beam reinforcing bars are passed through these holes and are fixed with grouting mortar. The structuring method consists of standing the columns upright at the site and attaching precast beams to them.

To be able to apply such precast concrete subassemblages to actual structures it is necessary to study the following points to confirm satisfactory seismic performance.

(1) Beam-column subassemblages integrated by grouting high-strength mortar should have same stiffness and strength as similarly designed structures cast monolithically.

(2) Reinforcing bars fixed inside the joint show no slippage due to bond deterioration and the entire structure possesses substantial ductility.

In order to study these items, experiments were conducted assuming (a) application of precast concrete subassemblages of Type 1 to a structure of beam yielding type, (b) application of precast concrete assemblages of Type 1 to a structure in which a plastic hinge occurred in a column at one side of the joint, and (c) application of precast concrete subassemblages of Type 2 to a structure of beam-yielding type.

EXPERIMENTAL PROGRAM

Test Specimens

Experiments were conducted in three series. The specimen configurations in each series are shown in Fig. 1. Descriptions of the specimens are given in Table 1.

The specimens in Series A consisted of three pieces and were of full size. Reinforcement details of specimens are shown in Figs. 2-4.

Specimens A1 and A2 were exterior joints and the columns were octagonal in shape. Specimen A3 was an interior joint and the columns were square in shape. A1 was a cast in place specimen, while joints and beam in A2 and A3 were precast concrete members. All of the specimens had transverse beams. The three specimens were designed so that yielding of beams would occur first, with strengths of columns and joints designed to be higher than ultimate flexural strengths of beams. Beam reinforcing bars of A1 and A2 were anchored bent inside joints, the total lengths being 30 $_Gd_b$ from beam end to bar ends ($_Gd_b$: diameter of beam reinforcing bar). In case of A3 the beam bars continued through the joint with anchorage length being 27 $_Gd_b$ to 34 $_Gd_b$ depending on reinforcing bar sizes. For shear strength of the joint to be adequate, steel plates were used as shear reinforcement. Specimens A2 and A3 had sleeve pipes in the joints for column reinforcing bars to go through and the positions of these pipes were restrained by the steel plates.

Specimens of this series were designed so that plastic hinges would develop at beam ends. Tensile stresses of the main column reinforcing bars fixed inside sleeve pipes with grout were kept below yield. The sleeve pipes were the same as those used for tendons in prestressed concrete work with inside diameters at least equal to reinforcing bar diameters plus 2.2-2.5 cm. In Series A, compressive strengths of concrete were high, exceeding 500 kgf/cm^2, for both precast and cast-in-place concrete.

Specimens of Series B consisted of two pieces and were of 1/2 scale. Details of the test specimens are shown in Fig. 3. Beams and joints were precast members in both specimens. B1 was a specimen designed for a plastic hinge to be produced at the beam end as for Series A, but concrete strength of precast members including joints was lower ($f'c$ = 248 kgf/cm^2). B2 was a specimen of column yielding type which yielded at the column head of the lower column. Both B1 and B2 has beam reinforcing bars continuing through the joints with $D_c/_Gd_b$ = 26.6-32.7 (D_c : column depth) for B1 and $D_c/_Gd_b$ = 21.1 for B2. Since B2 was a column-yielding type it was thought slippage of beam reinforcing bars would not occur. The column reinforcing bars of B2 were D16 with $D_G/{}_c d_b$ = 23.4 (${}_c d_b$: diameter of column reinforcing bar, D_G: beam depths), but these column bars were fixed with mortar in sleeve pipes of 38 mm inside the joint. Steel plates were used for shear reinforcement for the joint which also served to fix the sleeve pipes inside the joint.

Specimens for Series C consisted of three pieces and were of full size. Details of the specimens are shown in Fig. 4. For this series, unlike for Series A and B, sleeve pipes in the joints were installed in the horizontal direction and beam reinforcing bars were continued through these sleeve pipes and grouted for anchorage. Moreover, the specimens were designed to yield in flexure, with plastic hinges developed at beam ends. Beam reinforcing bars of specimen C1 continued through the joint with $D_G/{}_c d_b$ as 20. Since compressive concrete strength of the monolithic subassemblage was about 300kgf/cm^2, it was expected that slippage of beam reinforcement would occur inside the joint after yielding. Specimens C2 and C3 were made with the aim of preventing the kind of slippage occurring in C1 by fixing beam bars in the sleeve pipes inside the joints with high-strength mortar. Specimen C2 used sleeve pipes of lengths equal to column depth. Specimen C3 used sleeve pipes 59.6 cm in length and shorter than column depth. The sleeve pipes confined by steel plates served concurrently as shear reinforcement for the joint. The sleeve pipes used were of circular diameter 60 mm. Ordinarily, specimens C2 and C3 could have been made as precast subassemblages with columns and joint cast monolithically, but since it was not necessarily required that columns and joints be precast for the purpose of these experiments, which was to prevent slippage of beam reinforcement in gross deformation after yielding, the specimens were made placing concrete monolithically for the beams, columns, and joint. With C2 and C3, beam reinforcing bars were passed through sleeve pipes beforehand and were subsequently grouted with high-strength mortar. Reinforcing bars were placed so that these parts would be accommodated at their designated positions inside the joints.

The properties of the concrete and grouting mortar used in the specimens are given in Table 2. All concrete was supplied in the form of ready-mix concrete. The properties of the reinforcing bars and steel plates used are given in Tables 3a to 3c.

Test Setup

The test setup for the full-scale specimens of Series A is shown in Fig. 5. Specimen were laid down horizontally and placed on ball bearings of low frictional resistance. Steel blocks were fixed on the testing floor and loading was done using these blocks for reaction. The top and bottom column ends were roller-supported and loads were applied to beam ends in a state of a given axial force applied to the column.

Experiments for Series C were also conducted with specimens in a horizontal position similarly to Series A.

With Series B, since the specimens were of half-size and small, large loads were not required as for the specimens in Series A and C, so the experiments were carried out using steel frames for loading.

Testing Procedure

In Series A and B, and axial compressive stress of 48 kg/cm^2 was initially placed on the columns and maintained throughout the test. This stress was kept low because it tends to work advantageously in maintaining the integrity of column and joint. Axial forces were not applied to the columns in Series C.

Repetitive loads were applied to the specimens in the sequence shown in Fig.6. Except for Specimen B2, displacement at the time of yielding of beam reinforcing bars was taken as δy. With this as the basis, loads were repeated two cycles each at $2\delta y$, $3\delta y$ .. $6\delta y$. For series C, loading at the two cycles for $5\delta y$ was omitted. With Specimens A1 and A2 of Series A, loading at two cycles for $8\delta y$ was done after completing the cycles for 6 y.

In the case of Specimen B2, because it was column-yielding type, the deformation at time of yielding of column bars in the lower column was taken as δy.

TEST RESULTS

Failure Process

The ultimate failure modes of the various specimens are shown in Fig.7.

For every specimen in Series A, flexural failure occurred at beam ends. Column reinforcing bars did not yield. It appears that shear cracking did not occur at joints because of confinement provided by tranverse beams. Construction joints between columns and precast concrete joint panels in Specimens A2 and A3 were sound until testing was finished. After completion of the tests, the mid-height portions of joints were cut with a diamond saw. As a result of examination of grout around main column reinforcing bars, it was confirmed that the condition of grout had been good and cracking had not occurred in either of the specimens.

With Specimen B1 in Series B, flexural and flexural shear cracks at beam ends, flexural cracks at column ends, and diagonal cracks at the joint occurred during the first loading cycle following which beam reinforcing bars yielded. Bond deterioration of beam bars occurred inside the joint. After yielding in flexure, bond failure occurred inside the joint.

With Specimen B2, after flexural cracks had occurred at the head of the lower column, beam ends, and base of upper column, diagonal shear cracking occurred inside the joint. As B2 was designed for column yielding, none of the beam bars yielded to repetitions of 6δy. Beam bars were observed to yield during loading to ultimate failure. From the cycles at 4δy, opening of cracks at construction joints and lateral dislocation between the joint and lower column became prominent. In the end, flexural failure of the lower column occurred.

In Series C, bond failures accompanied by pullout of beam bars from joints occurred after beams had yielded. Pullout of beam reinforcing bars of Specimen C2 in which these bars were fixed with grout inside sleeve pipes of the same lengths as column depth occurred chiefly due to slippage at the interfaces between inner surfaces of sleeve pipes and grouting mortar but occurred also at boundaries between outer surfaces of sleeve pipes and concrete. In the case of Specimen C3 with the sleeve pipes confined by steel plates, the confining steel plates bulged out substantially toward the direction of pullout force of beam reinforcing bars. The amount of bulging of steel plates at the time of completion of testing was approximately 12 mm at the center of the column.

Figs. 8a) and b) show the column reinforcing bars inside the joint panel of Specimen B2 exposed by chipping away concrete after completion of tests. The conditions as enclosed by sleeve pipes are shown in Fig. 8a). At parts close to the head of the lower column, the grout mortar inside the sleeve pipes had been crushed, while the condition on removing the crushed concrete after stripping sleeve pipe is shown in Fig. 8b). In the case of this specimen, the column bars were subjected to large repetitive stresses extending into the plastic range at the column head and the high-strength grouting mortar in the sleeve was crushed because of this. The maximum extent of crushing was approximately 12 to 15 cm.

Initial Stiffness

The calculated and measured values of initial stiffness of the various specimens are given in Table 4. The calculated stiffness of the prefabricated specimens were obtained assuming the specimens to be monolithic and elastic. The ratios between calculated and measured values are in a range of 0.77 to 1.07. The calculated values agree well with the measured values except C2. It was ascertained that specimens made with beams and joints as precast concrete subassemblages joined with columns using grout mortar had initial stiffness similar to specimens made monolithically.

Load-Displacement Curve

Load-displacement curve are shown in Fig. 9. The loads on interior joints are average values of loads on beams at the two sides, while displacements are the sums of displacements at the loading points of the beams at the two sides. The displacements at the two sides indicated almost same values in case of Series C because bond failures occurred before beam ends had largely damaged. In the specimens of which beam ends had largely damaged in Series A and B, the difference was observed after large deformation between the displacements of two sides: (for A3, difference of the two was less than 1.6% before deformations of 5δy but it became 16% at the cycle of 6δy because one side of beam ends had damaged).

All three specimens of Series A showed favorable hysteresis loops, with restoring force characteristics indicating no strength decline and no slipping phenomena of beam reinforcing bars to gross deformations of 6δy (drift angel : 3.05-3.27 %). No difference was noticeable between a joint of precast concrete construction and one of cast-in-place construction.

In Series B, Specimen B1 of beam-yielding type indicated good restoring force characteristics with no strength decline to 4δy, but in repetitions at 5δy and 6δy, pinching due to pullout of beam bars from the joint occurred and strengths were also lowered. With Specimens B2 of column-yielding type, satisfactory restoring force characteristics with no strength decline were exhibited for deformations to 6δy.

With Series C, Specimen C1 of ordinary reinforcing bar arrangement exhibited no strength loss up to repetitions of 2δy, but subsequent hysteresis loops indicated pinching due to bond deterioration of beam bars inside the joint, while strength decline also occurred. Specimens C2 and C3 with main reinforcing bars of beams grouted in sleeve pipes inside joints did not show strength declines in repetitions up to 3δy, indicating slightly improved performance compares with C1. However, the hysteresis curves indicated pinching and slipping phenomena due to bond deterioration in the joints. There was hardly any difference in the deformation properties of C2 and C3: improvement was not recognizable with grouted portions confined with steel plates in the case of C3.

Shear Distortion of Joint

The shear distortion of the joint in Specimen B2 is shown in Fig. 10.

The proportion of shear distortion at positive maximum load was approximately 22%. The shear distortion and shear stress of B2 were larger than those of other specimens.

The maximum shear distortion of B1 was about one third of B2. The maximum shear distortion of A3 was much smaller than B1 and about one tenth of B2 because the specimen had transverse beam. The hysteresis loop of shear distortion of joints of Specimens A1, A2 shows almost elastic response because of transverse beams.

The joint shear force and the shear stress intensity at maximum load are given in Table 5. The shear stress intensities of the joints were 33-35 kgf/cm^2 in exterior joints, and 69-79 kgf/cm^2 in interior joints when omitting Specimen B2, approximately double the values for exterior joints. The joint shear stress intensity of the column-yielding type Specimen B2 at maximum load was the greatest.

Maximum Strength and Yield Strength

Measured and calculated values of maximum strength and yield strength are given in Table 6. In the calculation, concrete compressive stress blocks were expressed by exponential functions while reinforcing bars were modelled with bilinear stress-strain curves. The ${}_{T}P_{Y}/{}_{C}P_{Y}$ (${}_{T}P_{Y}$, ${}_{T}P_{Y}$: see foot notes of Table 6) values of yield loads ranged from 0.94 to 1.08 and there was good agreement between measured and calculated values. The ${}_{T}P_{MAX}/{}_{c}P_{MAX}$ (${}_{T}P_{MAX}$, ${}_{C}P_{MAX}$: see foot notes of Table 6) values of maximum loads were in a range of 1.08 to 1.38. The measured values of specimens that showed flexural failure were larger than the calculated values, because the reinforcing bars underwent strain-hardening at gross deformation

Relative Slippage of Precast Concrete Joint and Column

The elastic stiffnesses of prefabricated specimens were shown to be equal to those of specimens made monolithically. A question remained of what would happen in case stresses of a column of a specimen were to enter plastic ranges.

The relative slippages of the grouted portions were measured for Specimens B1 and B2 of Series B. The results are shown in Fig. 11 with the instrumentation for displacement. With Specimen B1 of beam-yielding type in which column ends did not yield, the relative slippage between joint and column was very small at 0.2 mm up to the cycles for 6δy. On the other hand, with B2 which yielded at the head of the lower column, there were relative slippages of 0.4 mm at δy and more than 6 mm at 3δy. If the construction joint like B2 is made at the portion where a plastic hinges is produced, large deformation due to slippage will occur at that part.

Bond Stress Intensity of Reinforcing Bar Grouted in Sleeve Pipe

The bond stress intensities of beam reinforcing bars or column reinforcing bars in joints are shown in Figs. 12 and 13. Both C2 and B1 were specimens of beam-yielding type. But C2 was a specimen in which beam reinforcement yielded at both ends, right and left, of horizontal sleeve pipes, and B1 was a specimen in which column reinforcement did not yield at the top and bottom ends of vertical sleeve pipes. The strains inside the joints were measured cutting grooves of small widths at the sides of longitudinal ribs of deformed bars and attaching strain gauges.

Figs. 12a), 12b) show the bond stress distributions of the top and bottom beam bars in Specimens C1 and C2. Large differences did not exist between C1 of ordinary reinforcing bar arrangement and C2 of grouting inside sleeve pipes. The bond stresses were 26 to 78 kgf/cm^2 for C1 and 26 to 81 kgf/cm^2 for C2. Fig. 13a) shows the bond stresses at ② and ③ of the top bars of Specimens C1 and C2 at maximum loads of the individual cycles in order from δy to $6\delta y$. It is noted that bond deterioration of ordinary beam reinforcement becomes larger than that of grouted reinforcement as deformation of the specimens becomes larger. In contrast, Fig. 13b) shows the bond stresses of column bars of B1 which did not yield at the column end, and as a matter of course, the bond stresses at the various parts were small, with bond stresses increasing as deformation progressed, and no sign of bond deterioration can be seen.

Load-Strain Curve of Reinforcing Bars in Joint

The load-strain curves of beam reinforcing bars inside the joints of Specimens A3 and B1 are shown in Fig. 14. Both were specimens of beam-yielding type, but A3 failed in flexure, while B1 showed a decrease in strength with slippage of reinforcing bars in joints occurring after yielding in flexure. Firstly, yielding occurred at beam end, followed by yielding at an inner location when deformation was $3\delta y$. In case of Specimen A3, strain at the center of the joint remained in the elastic range. With Specimen B1, strain was in the elastic range until $3\delta y$, but when loading to $4\delta y$, it became impossible to make measurements with the strain gauge, and it is assumed that loosening of bond had occurred at that location. There were no large differences in the load-strain curves of beam reinforcing bars in both specimens.

The load-strain curve of column bars inside the joint of Specimen B2 of column-yielding type are shown in Fig. 15. It is shown that yielding occurred first at the column end with the yielded zone gradually extending inward.

Anchorage Length of Beam Reinforcing Bars in Joint

Specimens A3, B1, and C1 were cases of interior joints, with A3 failing in flexure having adequately large ductility; whereas for B1 and C1, beam reinforcing bars passing through the joints slipped after flexural yielding. These differences in failure modes and ductility were produced due to the differences in anchorage lengths of beam bars and concrete strengths in the joints. The $D_c/_Gd_b$-$f'c$ relationships of these three specimens are shown in Fig. 16. The curves in this figure are indicated according to the Design Guidelines of A.I.J.(3).

The largest beam bars of A3 and B1 were D32 and D16 with yield points of 3970 kgf/cm^2 and 3960 kgf/cm^2, respectively, (only a 10 kgf/cm^2 difference between the two). The solid-line curve in the Fig. 16 was obtained with fy = 3960 kgf/cm^2 and $\mu = 10$. The $D_c/_Gd_b$ for Specimen B1 was approximately the same as this curve, but the $D_c/_Gd_b$ for Specimen A3 was so large in comparison with the solid line. It was thought these differences affected failure modes and ductilities. As shown in Fig.9, hysteresis loop of B1 showed strength decline after deformations of $5\delta y$ due to bond deterioration of beam bars in the joint. But hysteresis loop of A3 did not show strength decline. For Specimen C1, a broken line was used in Fig.16. The $D_c/_Gd_b$ of Specimen C1 was lower than the recommended value in the Design Guideline(3). Judging by hysteresis loop of C1 in Fig.9, anchorage length of beam bars was insufficient. Here, μ = 10 was used in the equation, but it will be necessary to make μ slightly smaller.

CONCLUSIONS

The following conclusions may be drawn based on the results obtained with the program :

1. Type 1 precast construction (: sleeve pipes are installed vertically in the joint to pass through column bars), in which beams and joint are made into a prefabricated subassemblage with column reinforcing bars fixed in sleeve pipes inside the joint by grouting, can be applied to a beam-yielding mechanism structure. The stresses of column bars in the joint remain in the elastic range so that the behavior of the subassemblage is decided by the anchorage length of beam bars in the joint.
2. It is safer not to apply the precast construction of Type 1 to a structure in which beams do not yield and a column at one side yields. The lateral displacement at the boundary plane between joint and column becomes greater as deformation progresses, which has an influence over the deformation properties of the whole.
3. Type 2 precast construction (: sleeve pipes are installed horizontally in the joint to pass through the beam bars), in which columns and joint are made into a prefabricated subassemblage with beam reinforcing bars fixed in sleeve pipes inside the joint by grouting, should not be applied to a beam-yielding structure. Because the beam reinforcing bars inside the joint are subjected to larger repetitive stresses extending into the plastic range within hinge zones, slippage of the reinforcing bars will occur inside the joint, and ductility will not be improved as much as expected.

   Increasing sleeve pipe length to improve anchorage is meaningless because satisfactory behavior will be obtained by anchorage length of 25-30 $_Gd_b$ of beam bars without using sleeve pipes.
4. In applying Type 1 precast construction to an interior joint, an anchorage length of beam bars larger the $D_c/_Gd_b$ obtained by the estimation formula (assuming $\mu \leqq 10$) of the Design Guidelines of the A.I.J.(3) should be adopted. By designing in such a manner, beam column subassemblages will possess a larger ductility (4δy and greater) without slippage through the joint.
5. In applying Type 1 precast construction to an exterior joint, a favorable hysteresis loop will be developed even when subjected to repetitions of large deformation if anchorage

length of beam reinforcing bars is taken at 30 ${}_Gd_b$ from beam end to bar end. Straight and horizontal portion of anchorage should be taken more than 8 ${}_Gd_b$ and 0.5 D_c from beam end.

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Table 1 List of Specimens

series		A			B		C		
specimen		A1	A2	A 3	B 1	B2	C 1	C 2	C 3
beam	bxD (mm)	520x750	500x750	500x750	270x375	300x375	600x500	600x500	600x500
	top bars	2-D32 2-D25 2-D25	2-D32 2-D29 2-D29	2-D32 2-D29 2-D29	4-D16 2-D13	4-D16 2-D16	4-D35	4-D35	4-D35
	bottom bars	2-D25 2-D29 2-D25	2-D25 4-D29	2-D25 4-D29	2-D13 4-D16	2-D16 4-D16	4-D35	4-D35	4-D35
	stirrup p_w (%)	4-D13 @150 0.65	4-D16 @150 1.06	4-D16 @150 1.06	4-D10 @80 1.31	4-D13 @70 2.42	4-D13 @100 0.85	4-D13 @100 0.85	4-D13 @100 0.85
column	bxD (mm) or ϕ (mm)	octagon ϕ=950	octagon ϕ=950	850x850	425x425	400x400	800x700	800x700	800x700
	bars p_g (%)	12-D35 1.53	12-D35 1.53	12-D35 1.59	12-D19 1.91	12-D16 1.49	16-D35 2.73	16-D35 2.73	16-D35 2.73
	hoop p_w (%)	spiral D16 @100 double	spiral D16 @100 double	4-D16 @100 1.00	4-D13 @70 1.70	4-D13@70 1.81	4-D19@100 1.43	4-D19@100 1.43	4-D19@100 1.43
joint	shear rein-forcement ps (%)	PL12x100 ring-plate 1.11	PL12x100 ring-plate 1.11	PL12x100 band-plate 1.13	2PL-9x60 1PL-9x90 band plate 2.71	2PL-9x60 1PL-9x90 band plate 2.88	2PL-12x70 band plate 0.84	2PL-12x70 band plate 0.84	2PL-12x70 2PL-12x100 band plate 2.04
axial force N (tonf)		360	360	347	87	0	0	0	0
note		cast in situ	precast	precast	precast	precast	cast in situ	cast in situ	cast in situ

Table 2 Properties of Concrete and Grouting Mortar

series	specimen	position	concrete f_c' (kgf/cm²)	concrete E_c (10^5 kgf/cm²)	concrete f_t (kgf/cm²)	grout mortar f_c'' (kgf/cm²)
A	A1	upper column beam, joint	632	3.62	34.5	—
		lower column	636	3.56	34.2	
	A2 · A3	upper column	632	3.62	34.5	595
		beam, joint	583	3.67	37.8	
		lower column	633	3.68	38.7	
B	B1	upper column	496	3.12	31.1	643
		beam, joint lower column	248	2.18	25.9	
	B2	upper column	520	3.22	34.7	724
		beam, joint lower column	293	2.06	27.1	
C	C1	beam, joint, column	330	2.48	28.9	—
	C2	beam, joint, column	300	2.44	25.5	623
	C3	beam, joint, column	299	2.24	24.4	638

f_c'' =compressive stength of grout mortar

(10kgf/cm² = 98.1×10⁻² MPa = 0.142ksi)

Table 3 a) Properties of Bars (Series A)

bar size	fy (kgf/cm²)	Er (10^6kgf/cm²)	fu (kgf/cm²)	E. L. (%)
D35	4050	1.83	6460	19.5
D32	3970	1.75	6500	18.5
D29	4150	1.86	6270	22.7
D25	4010	1.81	6570	16.8
D16	3720	1.87	5440	18.1
D13	3930	1.88	5370	19.3

Table 3 b) Properties of Bars and Steel Plate (Series B)

bar size	fy (kgf/cm²)	Er (10^6kgf/cm²)	fu (kgf/cm²)	E. L. (%)
D19	4050	1.94	5840	20.5
D16	3960	2.08	5840	19.9
D13	3870	1.84	5380	18.7
D10	4040	1.88	5500	21.5
PL-9	3420	2.10	4810	27.2

Table 3 c) Properties of Bars and Steel Plate (Series C)

bar size	fy (kgf/cm²)	Er (10^6kgf/cm²)	fu (kgf/cm²)	E. L. (%)
D35	4460	1.93	6570	21.7
D19	4060	1.78	5800	15.3
D13	3930	1.89	5640	19.9
PL-12	2750	2.12	4190	30.8

Table 4 Initial Stiffness

Specimen	K test	K calc	K test／K calc
A 1	135.0	131.0	1.03
A 2	118.0	127.0	0.93
A 3	47.6	52.9	0.90
B 1	17.1	17.9	0.96
B 2	110.3	103.3	1.07
C 1	60.0	63.2	0.95
C 2	48.5	63.2	0.77
C 3	59.4	63.2	0.94

1) K test = measured results unit(ton/cm)
2) K calc = P/δ_T, calculated results
P : beam load
δ_T : beam total deflection at loading point
3) $\delta_T = {}_b\delta_F + {}_b\delta_S + {}_c\delta_F + {}_c\delta_S + {}_P\delta_S$
${}_b\delta_F$: deflection caused by beam flexural deformation
${}_b\delta_S$: deflection caused by beam shear deformation
${}_c\delta_F$: deflection caused by column flexural deformation
${}_c\delta_S$: deflection caused by column shear deformation
${}_P\delta_S$: deflection caused by joint shear deformation
4) equivalent square column section was used in calculation of A1, A2 specimens

Table 5 Maximum Joint Shear and Joint Shear Stress

Specimen	Maximum Joint Shear (tonf)	Maximum Joint Shear Stress υ_p (kgf/cm²)	f_c' (kgf/cm²)	υ_p/f_c'	Calculated Joint Shear Strength (υ_u)		Comparison of Measured and Calculated value		Failure Mode
					υ_u·1 (kgf/cm²)	υ_u·2 (kgf/cm²)	υ_p/υ_u·1	υ_p/υ_u·2	
A 1	156	33.0	632	0.05	108	190	0.31	0.17	GB
A 2	163	35.1	583	0.06	108	175	0.33	0.20	GB
A 3	359	79.4	583	0.14	109	175	0.73	0.45	GB
B 1	75	73.1	248	0.30	141	110	0.52	0.66	GB−S
B 2	112	113.4	293	0.39	144	87.9	0.79	1.29	CB
C 1	251	69.6	330	0.21	107	99.0	0.65	0.70	GB−S
C 2	254	70.5	300	0.24	107	99.0	0.66	0.71	GB−S
C 3	248	68.8	299	0.23	123	89.7	0.56	0.77	GB−S

GB : beam flexural failure
CB : Column flexural failure
S : Slip of joint bars after beam yielding
f_c' : concrete compressive strength
υ_u·1 : Kamimura Equation
$= 95.1 + 0.5\, P_w\, {}_s\sigma_y$
P_w : shear reinforcement ratio of joint
${}_s\sigma_y$: yield strength of joint shear reinforcement
υ_u·2 : equation from Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on ultimate Strength Concept (Draft)
$= 0.3\, \sigma_B$
σ_B : concrete compressive strength

(10tonf=98.1×10³N=22.1 kips)
(10kgf/cm²=98.1 ×10⁻²MPa=0.142ksi)

Table 6 Yielding Load and Maximum Load

Specimen		${}_TP_y$ (tonf)	${}_cP_y$ (tonf)	${}_TP_y / {}_cP_y$	${}_TP_{max}$ (tonf)	${}_cP_{max}$ (tonf)	${}_TP_{max} / {}_cP_{max}$
A 1	+	39.8	36.9	1.08	47.1	39.4	1.20
	−	37.9	37.2	1.02	44.0	40.9	1.08
A 2	+	39.7	42.2	0.94	49.5	44.8	1.10
	−	40.5	40.2	1.01	48.2	44.1	1.09
A 3		41.9	40.3	1.04	52.2	43.5	1.20
B 1		10.5	10.2	1.03	11.8	10.8	1.09
B 2		20.5	18.9	1.08	28.5	20.7	1.38
C 1		42.7	41.8	1.02	46.5	42.8	1.09
C 2		40.7	41.6	0.98	47.0	42.5	1.11
C 3		43.5	41.6	1.05	45.9	42.5	1.08

($10\text{tonf}=98.1\times10^3\text{N}=22.1$ kips)

${}_TP_y$: average of left and right beams-end loads at beam bars yielding (for interior joint specimen) or average of left and right beams-end loads at column bars yielding (for specimen B2)

${}_TP_{max}$: average of left and right beams-end loads at maximum loading

${}_cP_y$: average of positive and negative beam-end loads at beam bar yielding (for interior joint specimen) or average of positive and negative beam-end loads at column bar yielding (for specimen B2)

${}_cP_{max}$: average of maximum positive and negative beam-end load (for internal joint specimen)

suffix T : measured value
C : calculated value

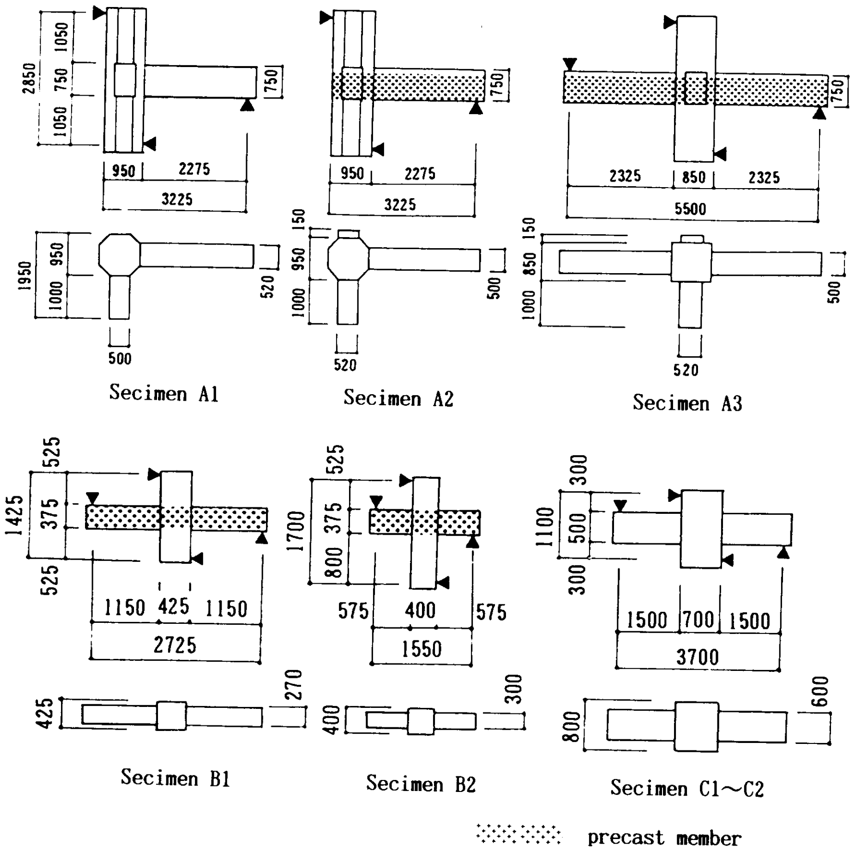


Fig. 1--Specimen configuration

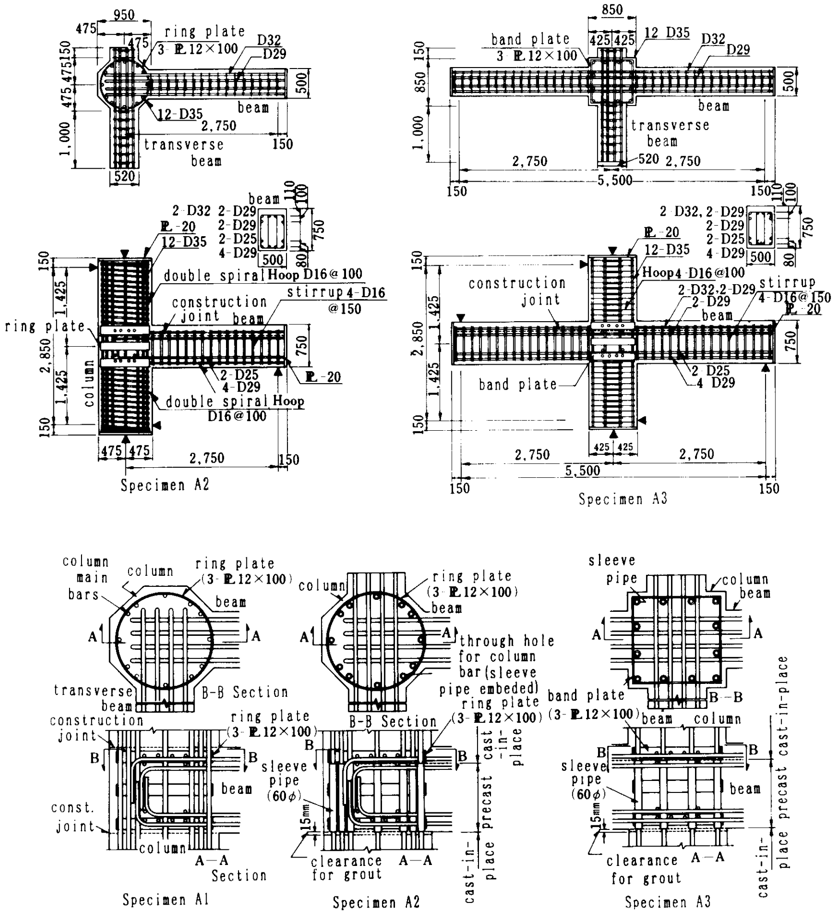


Fig. 2--Reinforcing details (series A)

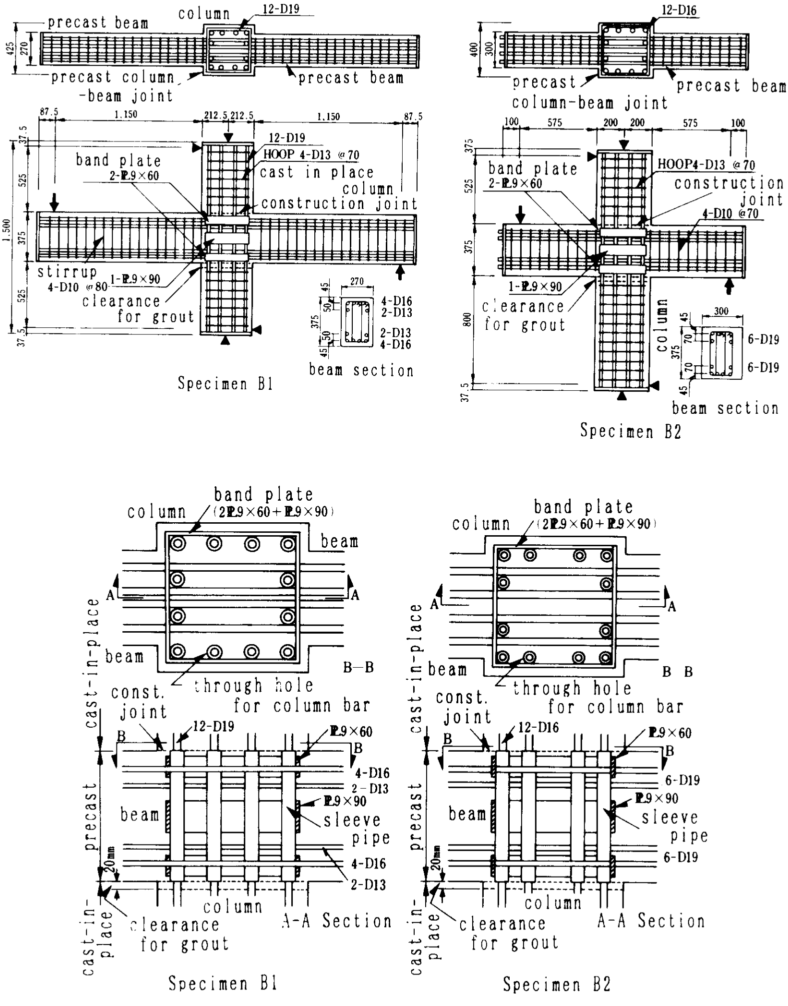


Fig. 3--Reinforcing details (series B)

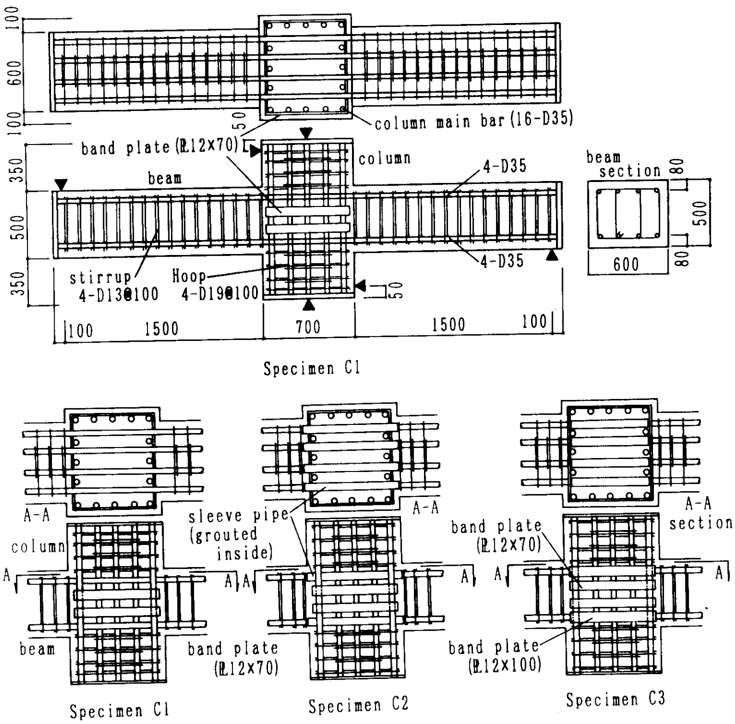


Fig. 4--Reinforcing details (series C)

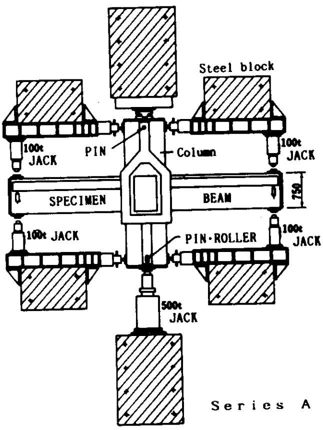


Fig. 5--Test setup

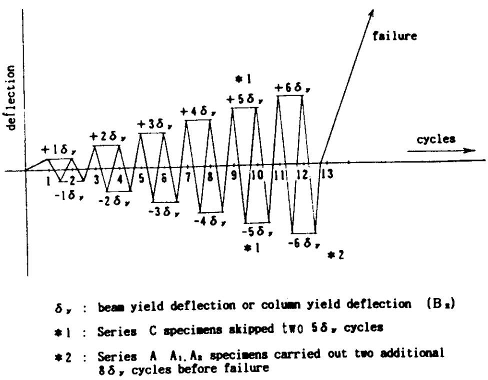


Fig. 6--Loading history

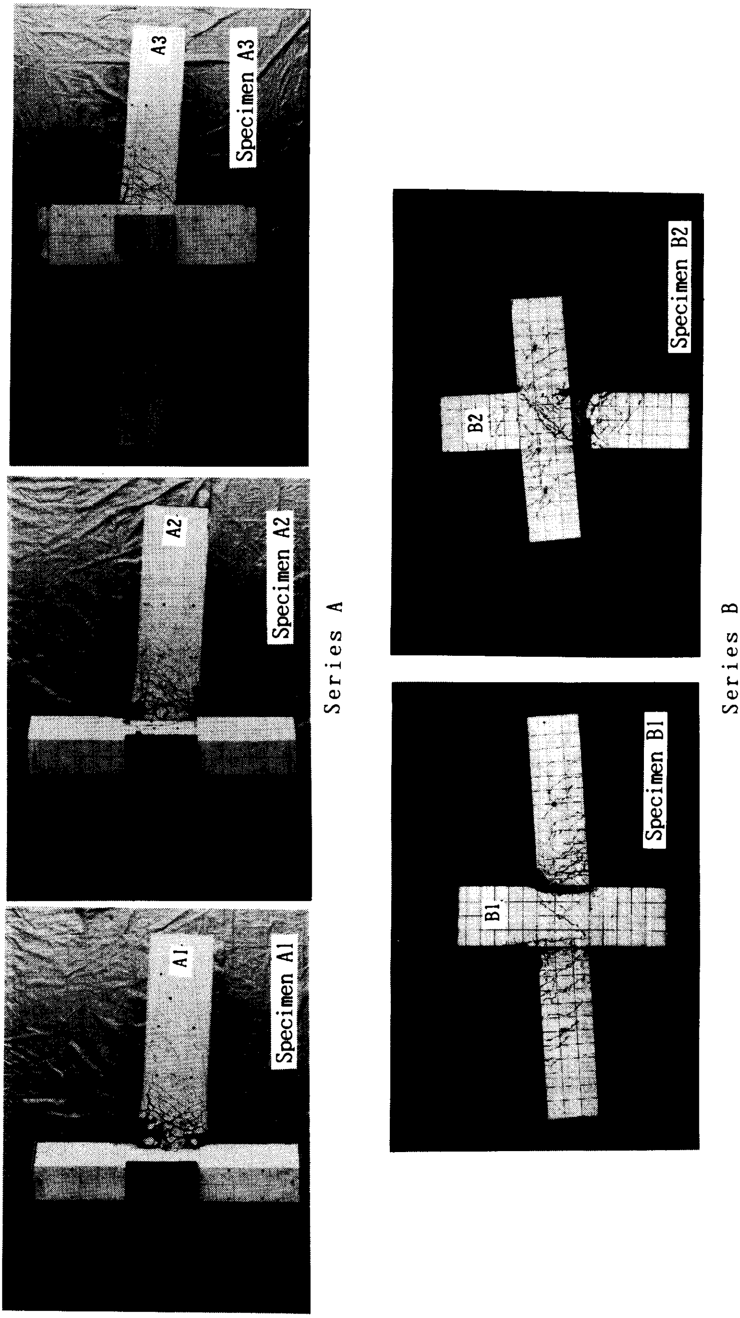
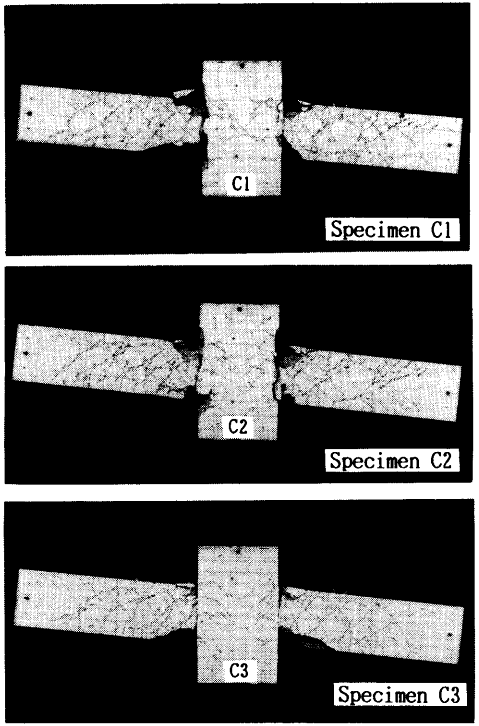


Fig. 7a--Failure pattern



Series C

Fig. 7b--Failure pattern

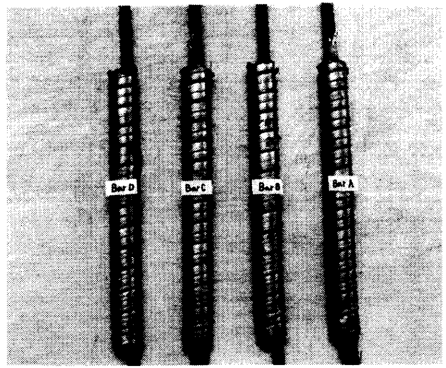


Fig. 8a--Column bars enclosed by sleeve in joint after loading

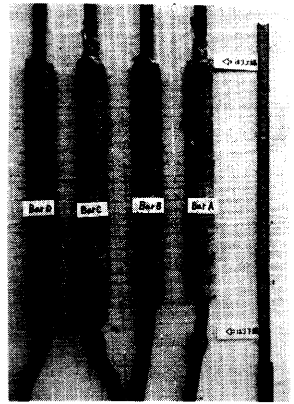
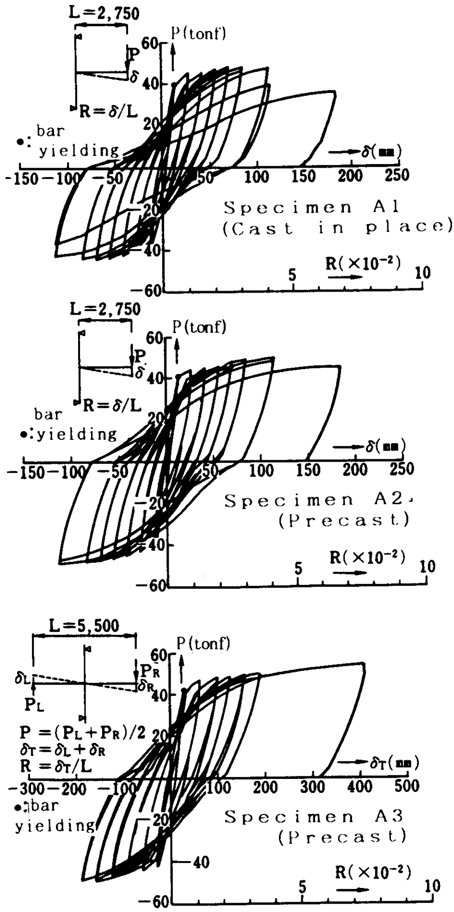
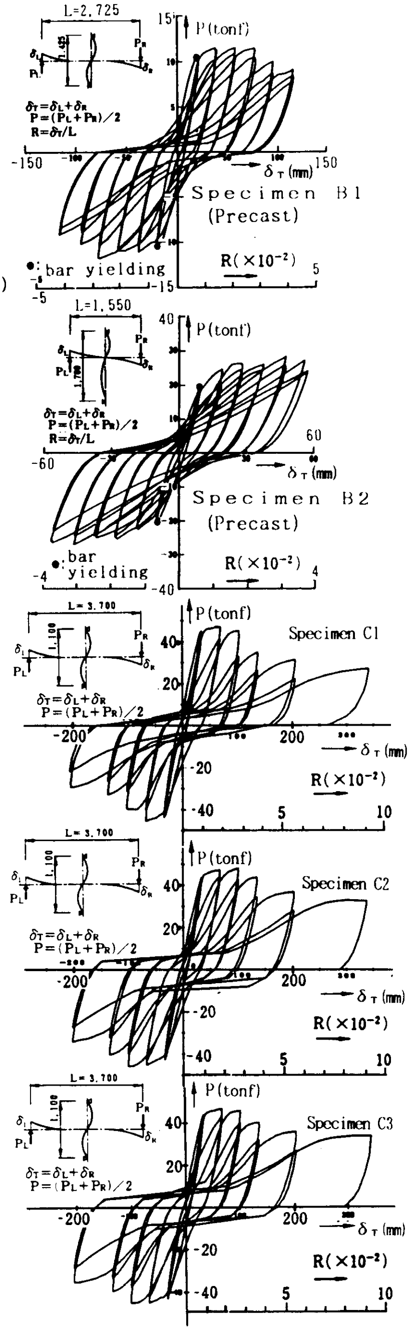


Fig. 8b--Column bars after stripping sleeves in joint after loading





10tonf= 98.1×10³N=22.1 kips
20tonf=196.1×10³N=44.1 kips
40tonf=392.3×10³N=88.2 kips
60tonf=588.4×10³N=132.3kips

Fig. 9--Load (P) ~ displacement (δ) relation

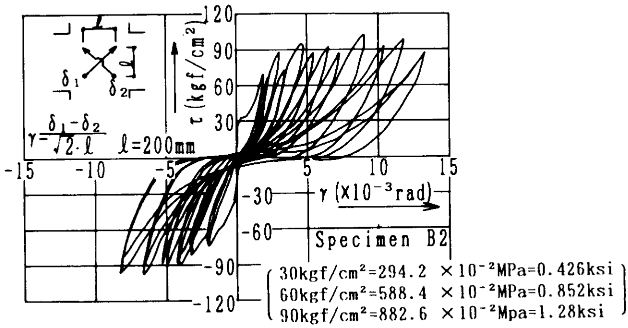
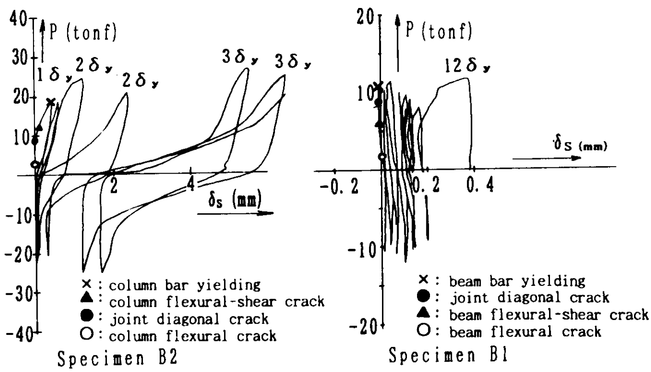
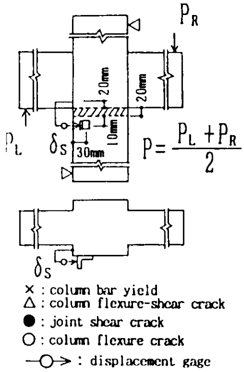


Fig. 10--Joint shear stress (τ) ~ shear distortion (γ) relation (specimen B2)





10kgf/cm² = 98.1 ×10⁻²MPa=0.142ksi
20kgf/cm²=196.1 ×10⁻²MPa=0.284ksi
30kgf/cm²=294.2 ×10⁻²MPa=0.426ksi

Fig. 11--Relative displacement of construction joint between joint and column

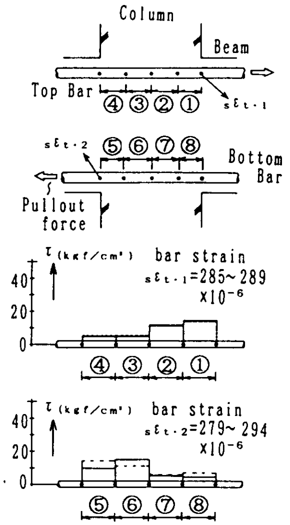


Fig. 12a--Bond stress distribution of beam bars in joint at low stress ($10kgf/cm^2 = 98.1 \times 10^{-2}MPa = 0.142ksi$)

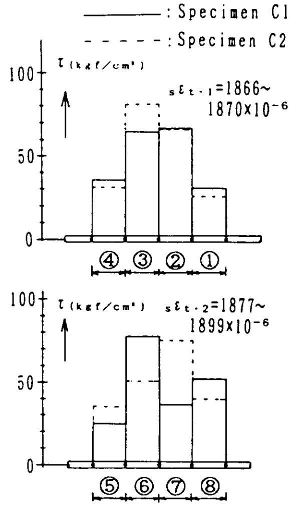


Fig. 12b--Bond stress distribution of beam bars in joint at high stress

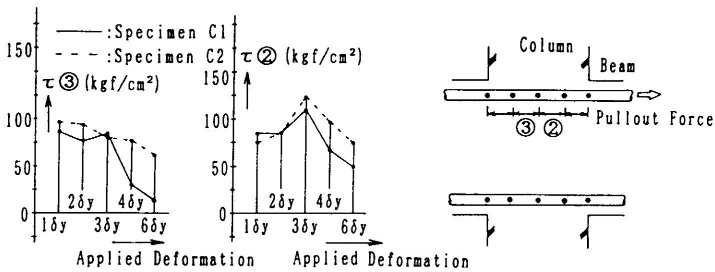


Fig. 13a--Bond stress of beam bars in joint

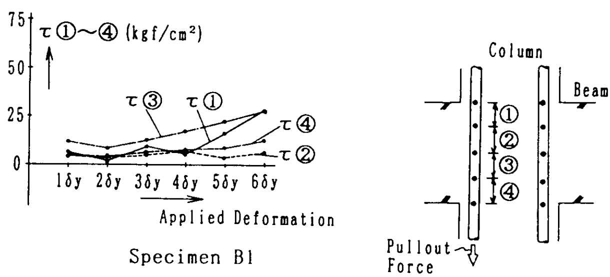


Fig. 13b--Bond stress of column bars in joint

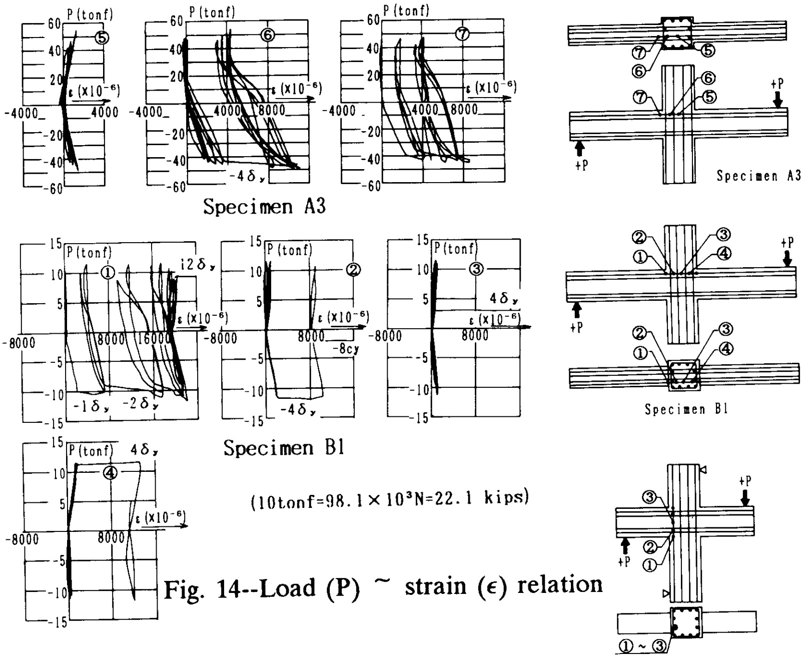


Fig. 14--Load (P) ~ strain (ϵ) relation

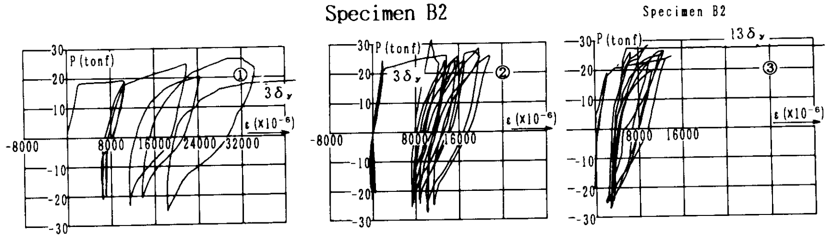


Fig. 15--Load (P) ~ strain (ϵ) relation (10tonf=98.1 x 10^3N=22.1 kips)

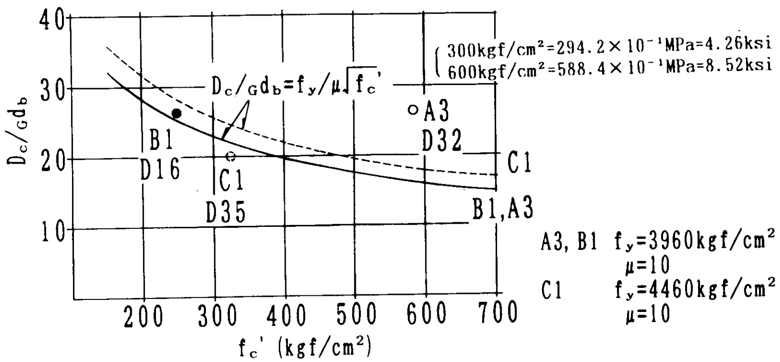


Fig. 16--Beam bar anchorage of interior joint

APPENDIX A: Two Structuring Methods Using Prefabricated Concrete Subassemblages

Two types of precast concrete construction as shown in Figs. A1 and A2 were devised as structuring methods for high-rise reinforced concrete structures.

Fig. A1 shows a method in which beams and joints are made into prefabricated subassemblages with holes in the vertical direction using sleeve pipes inside joints. Fig. A2 shows a method in which columns and joints are made into prefabricated subasemblages with holes in the horizontal direction using sleeve pipes inside joints.

In Type 1 of Fig. A1, column reinforcing bars are first arranged and the concrete is placed. The column reinforcing bars are protruding upward at this time. A prefabricated concrete subassemblage of beams and a joint is set on the column with the column reinforcing bars passed through sleeve pipes leaving a clearance of 1.5 to 2 cm on top of the column. High-strength grouting mortar is pumped in from this clearance and it is confirmed that the high-strength mortar reaches the top of the sleeve pipe. Connections with adjacent prefabricated subassemblages are made at middles of beams. Beam reinforcing bars are welded, stirrups are arranged, and concrete is cast in place.

In Type 2 of Fig. A2, short beam reinforcing bars are passed through sleeve pipes in the horizontal direction inside joints and are fixed by injecting high-strength mortar. These prefabricated subassemblages are erected in the field and precast beams are jointed to them. Concrete is cast in place at the connection parts.

APPENDIX B : Applicable Building

Views of prefabricated concrete subassemblages being erected at the 13-story reinforced concrete Lions Garden Kawaguchi in which the precast structuring method of Type 1 was applied are shown in Fig. B.

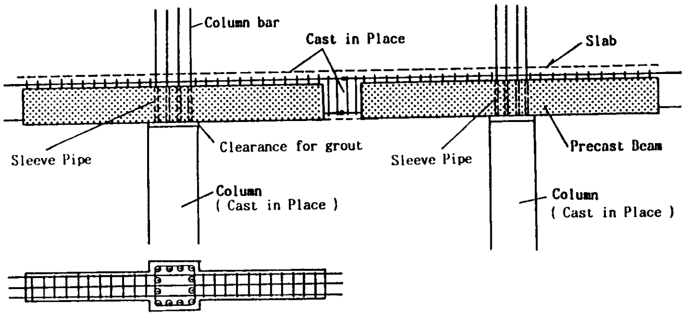


Fig. A1--Precast concrete construction patterns of column-beam joint (type 1)

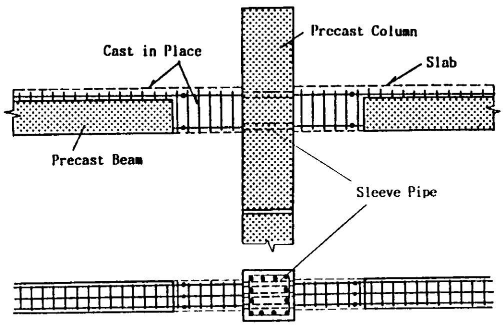
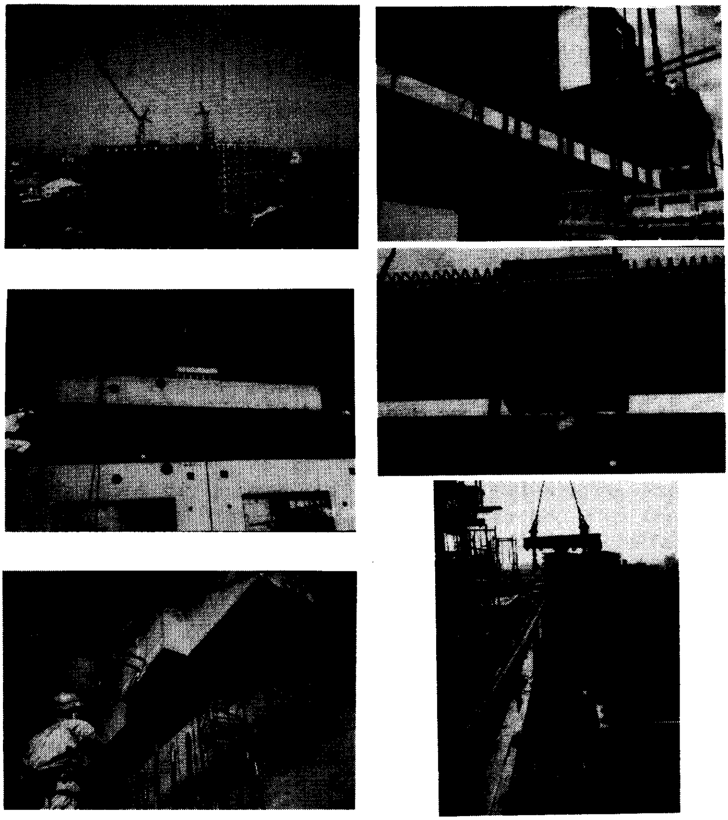


Fig. A2--Precast concrete construction patterns of column-beam joint (type 2)



- The specified concrete strengths of this building were 240-360kg/cm².
- The specified yield strength of reinforcing bars used in beams and columns of this building was 3500 kg/cm². Partially reinforcing bars of 4000 kg/cm² specified yield strength were used.
- Reinforcing bar sizes of columns were D32 (32mm diameter) and D35 (35mm diameter).
- Reinforcing bar sizes of beams were D32, D35 and D38 (38mm diameter).
- The same grouting mortar as described in this program was used.
- Similar steel band plates shown in Figs. 2-4 were used in joints for shear reinforcement and fixxing sleeve pipes.
- As high strength mortar exceeding 500 kg/cm² was grouted in the sleeve pipes in joints, longitudinal column bars were confined by this steel band plates though plates did not contact directly with column bars.

Fig. B--Construction of Lions Garden Kawaguchi Building