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Mixed Structural Systems of Precast Concrete Columns and Steel Beams

Systèmes structuraux mixtes de poteaux en béton prémoulé et poutres en acier

Mischbausysteme aus Betonfertigteilstützen und Stahlträgern

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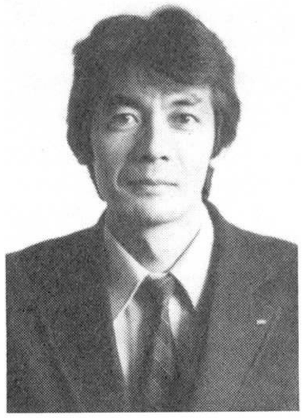

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SUMMARY

In this paper, mixed structural systems which consist of composite precast concrete columns and steel beams are examined through the results of an experimental research program in which interior beam-column joint specimens were tested under reversed cyclic loading.

RÉSUMÉ

Les systèmes structuraux mixtes, consistant en poteaux de béton prémoulé et poutres en acier, sont étudiés dans le présent document au moyen des résultats obtenus lors de l'exécution d'un programme de recherche expérimental, au cours duquel spécimens de joint intérieur poutre-poteau ont été soumis à des essais sous une charge cyclique inversée.

ZUSAMMENFASSUNG

Diese Abhandlung befaßt sich mit der Untersuchung von baulichen Mischsystemen, die aus vorgefertigten Verbundstützen und Stahlträgern zusammengesetzt sind. In eine experimentellen Forschungsprogramms wurden Rahmenknoten unter Wechselbeanspruchung untersucht.

1. INTRODUCTION

In Japan, a type of composite structural system called the steel reinforced concrete (SRC) structural system has been popular for many years. Since the early 1980s, mixed structural systems composed of steel reinforced concrete (SRC) columns and steel beams have been studied. Recently, new mixed structural systems composed of reinforced concrete (RC) columns and steel beams were developed.

In addition, development of precast concrete frame systems, which simplify the assembly of reinforced concrete structures, has long been a major issue in the advancement of construction methods. Due to severe conditions caused by earthquakes in Japan, precast units have been joined together at the center of the beams and columns where a minimal degree of stress and deformation is expected. However, this leads to several disadvantages including complicated configuration, high transportation costs for the large units, and limited applicability. Therefore, we developed a more simplified precast concrete frame system, as shown in Fig. 1. In this system, which was first implemented in Japan in 1978, the basic units consist of precast concrete columns and beams. The column units are separated from the beam units. The column and beam units are joined at the beam-column joint by cast-in-place concrete. This system has since been applied to many types of large-scale buildings, such as shopping centers, schools and hospitals.

Further development of new mixed structural systems which consist of composite precast concrete columns and steel beams has been carried out along the same line as that of the first system, as shown in Fig. 2. Aspects which must continue to be improved include the stress transfer mechanism at the beam-column joint and the seismic capacity of the joint between precast concrete units. To clarify the stress transfer mechanism, and to verify the effectiveness of our systems, we carried out experimental tests of the interior beam-column joint subject to reversed cyclic loads similar to the effect of an earthquake. Six specimens consisting of approximately two-thirds-scale models, whose failure modes were of the beam collapse and joint collapse types, were adopted. From the experimental results, a stress transfer mechanism between the composite precast concrete columns and steel beams in the present systems, and the ductility of the beam-column joint, were studied.

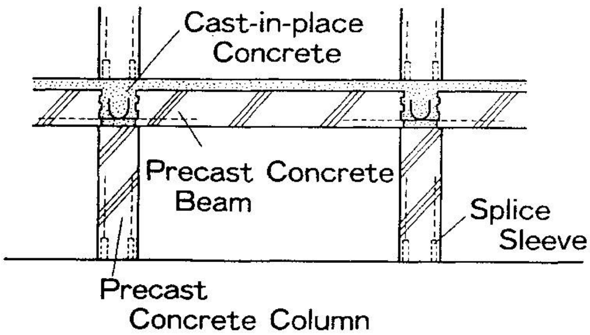

Fig. 1 Precast Concrete Frame System

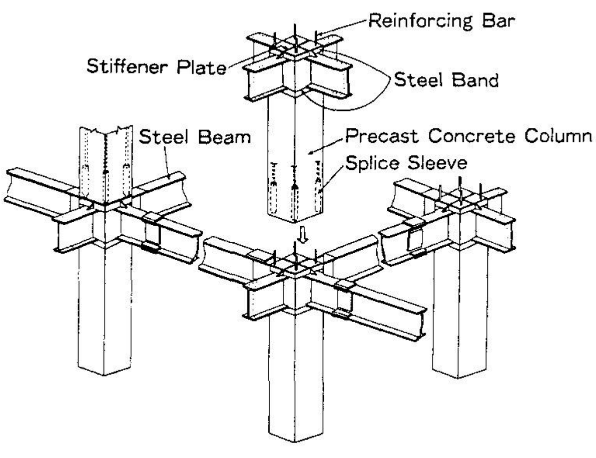

Fig. 2 New Mixed Structural system

2. JOINT DETAILS

The present systems are classified into two systems in terms of the details of beam-column joints and precast concrete columns. In the first system, as shown in Fig. 2 and Fig. 3 (specimen NO. 2), the column consists of reinforced precast concrete enclosed by steel bands which are arranged at the top and the bottom of the column. The steel bands, whose width is equal to the thickness of the slab, are welded to the flanges of H-shaped steel beams. The steel bands are thought to be available for confining the concrete at the top and the bottom of the column. In the second system, the column consists of the reinforced precast concrete and tubular or H-shaped steel as shown in Fig. 3 (specimens NO. 3 and NO. 6). The steel, which is embedded in a reinforced concrete column, is welded to the steel beam. The embedded length of the steel is two times greater than the depth of the H-shaped steel or the diameter of the tubular steel. In both systems, mechanical joints of reinforcing bars, called splice sleeves, are embedded at the bottom of the columns.

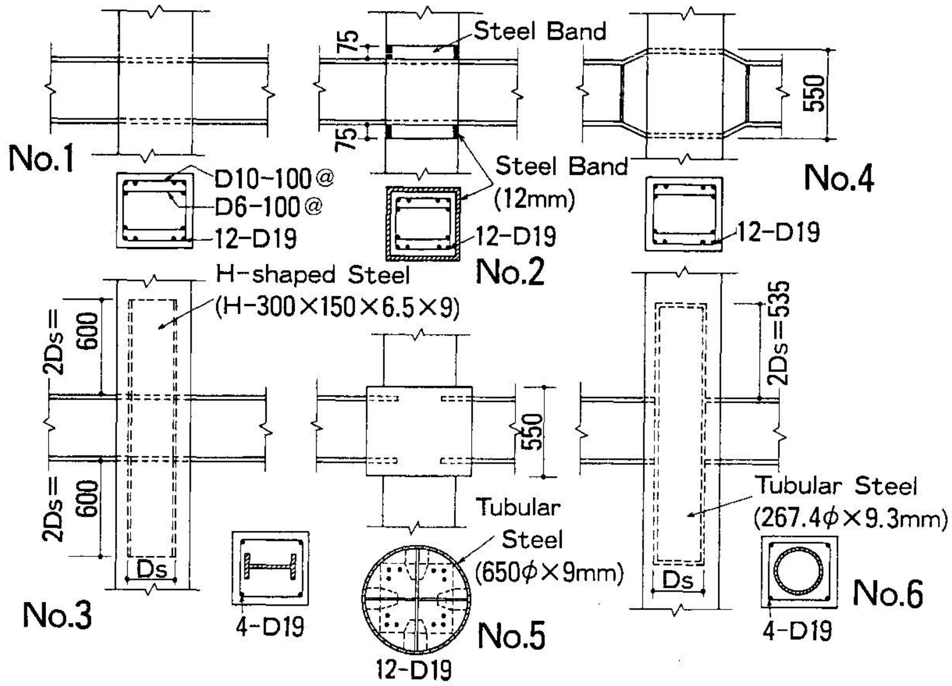

Fig. 3 Joint Details of Specimens

3. EXPERIMENTAL TESTS

3.1 Test Specimens

Six specimens consisting of approximately two-thirds-scale models were tested under reversed cyclic loads. A typical cruciform-shaped specimen geometry is shown in Fig. 4. Applied loads at the ends of the beams were so controlled as to give simultaneously the same magnitude but opposite sign displacement at loading points to these beams. A vertical load, which is kept constant ($\sigma_0 \fallingdotseq$ 4.0 MPa), is applied to the column in all specimens. In all specimens, the vertical column reinforcement was designed to exceed the anticipated beam strength. In addition to monitoring the applied beam loads, story drift and deformation of each beam, column and joint, we measured steel and concrete strains. The dimensions and material properties of the column and beam in each specimen are listed in Table 1 and Table 2. The H-shaped steel beam, which is common to all specimens, is used in the tests. The plate dimensions used in the beams are as follows: web plate 9 × 400 mm; flange plate 12 × 150 mm. The dimensions of the column section, which is common to all specimens, are 450 × 450 mm.

Specimens NO. 1, NO. 2, NO. 4 and NO. 5 are composed of reinforced precast concrete columns and steel beams. Specimens NO. 3 and NO. 6 are composed of tubular or H-shaped steel and reinforced precast concrete columns and steel

Table 1 List of Specimens

Specimen	Column Section	Column Steel Portion	Column Reinforcement Vertical	Column Reinforcement Hoop	Beam Section	Panel Steel Hoop
NO.1	450x450		12-D19	D10-@100 D6- @100	H-400x150 x9x 12	Web Plate 12mm Hoop D10- @80
NO.2		Steel Band (75x12mm)				
NO.3		H-300x150x6.5x9	4-D19	D10-@100		
NO.4			12-D19	D10-@100 D6- @100		Web plate 12x550 mm Hoop D10- @80
NO.5						Tubular steel(650 ϕ) No Hoop
NO.6		Tubular Steel (267.4ϕ x9.3mm)	4-D19	D10-@100		Tubular steel(267.4 ϕ) Hoop D10- @80

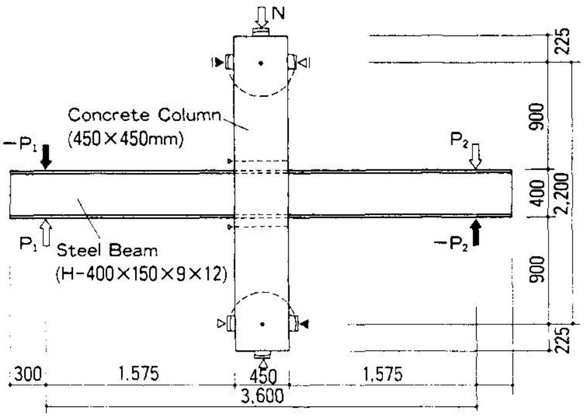

Fig. 4 Typical Test Specimen

Table 2 Properties of Materials

Material		σ_y (MPa)	$\sigma_{max.}$ (MPa)	E_s (MPa)	Elong.
Steel	6.5mm	312.6	489.0	1.92×10^5	.335
	9 m	285.7	462.0	1.95×10^5	.383
	12 mm	295.0	428.0	1.97×10^5	.360
Re-Bar	D10	364.6	522.0	1.77×10^5	.270
	D19	365.0	532.5	1.73×10^5	.208

Material		F_c (MPa)	f_t (MPa)	E_c (MPa)
Concrete	Panel	24.3	2.2	2.09×10^4
	Column	25.2	2.5	1.94×10^4

beams. In specimen NO. 1, the steel beam is continuous through the joint without special contrivances to increase the ductility of the beam-column joint. The dimensions of steel bands arranged at the top and the bottom of the column in specimen NO. 2 are 1.2 × 75 mm. In specimen NO. 5, the beam-column joint, the shape of which is cylindrical, as shown in Fig. 3 (NO. 5), is solid concrete enclosed by tubular steel. The dimensions of the tubular steel are as follows: diameter 650 mm; thickness 9 mm; height 550 mm. In specimen NO. 4, the field of the beam-column joint is extended. In specimens NO. 5 and NO. 6, the dimensions of H-shaped and tubular steel embedded in the reinforced concrete column are as follows: web plate 6.5 × 300 mm; flange plate 9 × 150 mm; tubular steel 267.4ϕ × 9.3 mm. The concrete is not solid inside the tubular steel in specimen NO. 6.

3.2 Test Results

3.2.1 Development of Test

The test results of all specimens are listed in Table 3. All specimens developed flexural yielding at the flange of the steel beam adjacent to the beam-column joint at the story drift angle of 0.7/100 ~ 1/100 rad. In specimens NO. 1 and NO. 4, which failed in panel shear, the deterioration of concrete at the joint and column adjacent to the joint was observed to be marked after the maximum load. In specimens NO. 2, 3 and 5, the decrease of bearing capacity was not observed until the story drift angle reached 5/100 rad. Specimen NO. 6 exhibited rupture of the steel beam flange at the story drift angle of 3/100 rad.

3.2.2 Concrete Cracking

In all specimens, flexural cracks in the column and diagonal shear cracks in

Table 3 Summary of the Test Results

Specimen	Flexural Strength of Beam (kN)			Shear Strength of Panel (MPa)			Mode of Failure
	Exp.(P)	Cal.(P_c)	P/P_c	$\tau_{exp.}$	$\tau_{cal.}$	$\tau_{exp.}/\tau_{cal.}$	
NO. 1	211(207)	190	1.11(1.09)	18.8(18.5)	18.6	1.01(0.99)	Beam yielding → Panel shear
NO. 2	253(255)		1.33(1.34)	23.4(23.8)		1.26(1.28)	Beam yielding
NO. 3	228(229)		1.20(1.21)	18.3(18.2)		1.11(1.10)	Beam yielding
NO. 4	245(251)	211	1.16(1.19)	14.7(14.3)		0.97(0.95)	Beam yielding → Panel shear
NO. 5	277(286)	203	1.36(1.41)	10.2(10.4)	12.0	0.84(0.86)	Beam yielding → Local Buckling of Flange → Flange Rupture
NO. 6	260(242)	190	1.37(1.27)	20.3(19.7)	23.8	0.85(0.83)	

( ); Negative Loading

the joint occurred at a story drift angle of 3/1000 ~ 5/1000 rad. Specimens NO. 5 and NO. 6 developed very few additional diagonal shear cracks in the joints after a story drift angle of 2/100 rad. In specimens NO. 1 and NO. 4, the shell concrete spalled in the four corners near and within the beam-column joint at a story drift angle of 4/100 rad.

3.2.3 Load-Deformation Response

The load-deformation curves of all specimens are shown in Fig. 5. The vertical axis indicates the applied beam load, which is proportional to beam moments adjacent to the beam-column joint. The horizontal axis indicates the story drift angle, which is a measure of the total relative angular rotation between the steel beam and precast concrete column. Specimens NO. 2, NO. 3, NO. 5 and NO. 6 showed a favorable spindle-shape hysteresis, but specimens NO. 1 and NO. 4 showed a pinching hysteresis shape under cyclic load reversals.

The contribution of the parts of each specimen to the story drift was estimated and is shown in Fig. 6. In specimens NO. 1 and NO. 4, the deformation of the beam-column connection markedly increased after a story drift angle of 3/100 rad. In specimen NO. 6, most of the deformation was contributed by steel beam deformation which led to the beam failure. In specimens NO. 5 and NO. 6, the connection deformation is very small. The connection deformation of specimen NO. 2 is slightly smaller than that of specimen NO. 3.

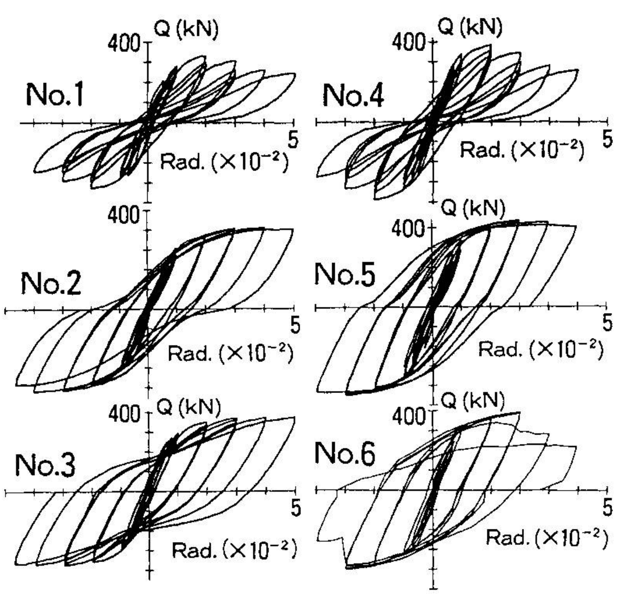

Fig. 5 Load-Deformation Curves

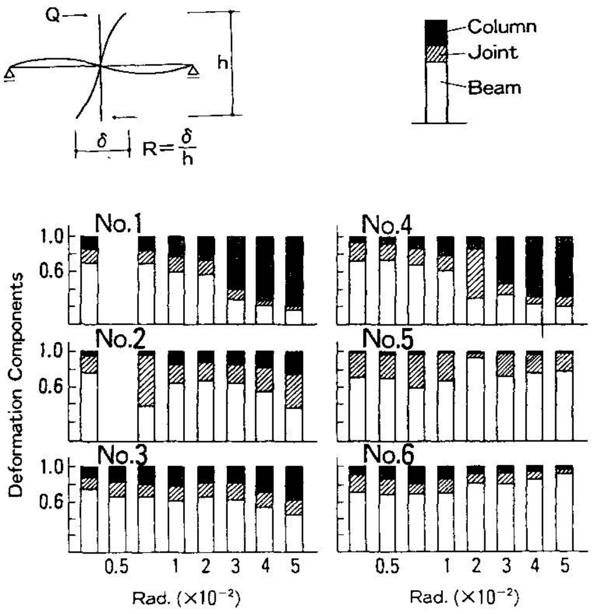

Fig. 6 Deformation Components

4. Stress Transfer Mechanism

Two hypothetic mechanisms for the stress transfer from the steel beam to the composite concrete column through a beam-column joint are shown in Fig. 7. Beam moments are shown as equivalent horizontal force couples acting in the beam flanges, and column moments are shown as vertical force couples. In mechanism TYPE I, the joint details of which are shown in Fig. 3 (Specimen NO. 2), the concrete compression field outside the steel beam is formed by a lever action of the beam flange and confinement effects of the steel bands. Steel bands are effective in preventing the spalling of the concrete in the four corners near and within the beam-column joint. In mechanism TYPE II, the joint details of which are shown in Fig. 3 (Specimens NO. 3 and NO. 6), the concrete compression field outside the steel beam is formed by the lever actions of the beam flange and the steel embedded in the concrete column. The test results indicate that it is impossible to transfer the stress by only a lever action of the beam flange, as in specimens NO. 1 and NO. 4.

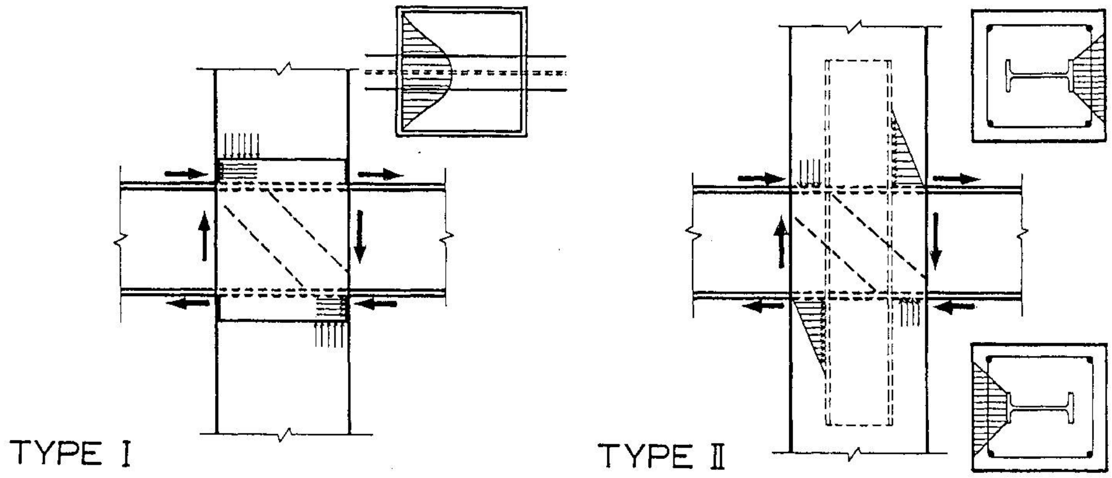

Fig. 7 Stress Transfer Mechanism

4. CONCLUSIONS

From the experimental results, the following conclusions can be stated.

1) A stress transfer mechanism between the precast concrete columns and steel beams in the present systems was studied.
2) The details of columns with steel bands or with tubular steel were very favorable for seismic design.
3) The beam-collapse-type specimens, which were designed for seismic loading, exhibited very satisfactory strength and ductility characteristics.

On the basis of these results, we refined the feasibility of these systems and their application to practical construction.

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