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STRUCTURAL BEHAVIOR OF COLUMNS AND BEAM-COLUMN SUBASSEMBLAGES IN A 30 STOREY REINFORCED CONCRETE BUILDING

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SUMMARY

A 30-storey ductile framed reinforced concrete structure has been designed using high strength concrete (design strength up to 420kgf/cm2) and additional core re-bars in the center of the exterior columns to resist earthquake loads.

This paper outlines results of several tests on exterior columns and beam-column subassemblages. These tests have been conducted to verify the strength and ductility of the new frame elements and the shear strength of the joint core concrete under earthquake-type loadings.

INTRODUCTION

In March 1987, a 30-storey reinforced concrete (R/C) framed structure heralded the birth of a new generation in Japanese high-rise R/C buildings.

The design of this building incorporated two major changes from that of previous high-rise R/C buildings in Japan (Ref.1). The innovations were the use of high strength concrete of up to 420kgf/cm2 (the concrete strength up to 360kgf/cm2 was used previously) and additional core re-bars in the center of the exterior columns (pre-stressing was applied previously to reduce tensile stress).

To verify the strength and ductility of the new frame elements, we have conducted the following two experimental investigations;

1. <u>COLUMN TESTS</u> : to determine the ultimate strength and ductility of exterior columns under large fluctuating axial forces (from 70% of the ultimate compressive strength to 60% of the ultimate tensile strength of the column).

2. <u>BEAM-COLUMN JOINT TESTS</u> : to verify a beam yielding mechanism of the frame subassemblages under earthquake type loadings and to determine the shear strength of the joint core concrete.

TEST PROGRAM

<u>OUTLINE OF THE BUILDING</u> Photo 1 shows the Park-City Shinkawasaki building, a 30-storey high-rise R/C condominium constructed in Kawasaki-City near Tokyo. A typical floor plan (29.8x29.8m) and an elevation (87.2m high) of the building frame are shown in Fig.1.

Photo 1 Park City Shinkawasaki Building

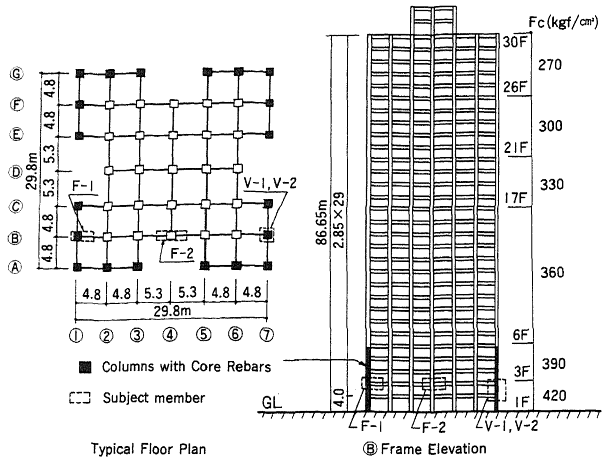
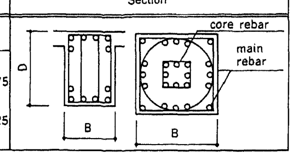


Fig.1 Building frame

All members of the building frame were cast in place. The high strength concrete (Fc=420kgf/cm2) was placed from the 1st to the 2nd floor. A set of additional longitudinal re-bars (core-rebars) was arranged in the exterior columns from the 1st to the 5th floor In order to resist large axial forces produced by overturning moments. .Table 1 shows typical dimensions and bar arrangements of the members. The maximum size of main re-bars was 41mm and their nominal yield strength was 40kgf/mm2.

Table 1 Typical members

	Exterior column				Interior column			Beam			Section
floor	B×D	main rebar	core rebar	hoop	B×D	main rebar	hoop	B×D	main rebar	stirrup	
30 ↓ 2	800×800 ↓ 850×850	12-D32 ↓ 16-D41	— ↓ 8-D41	D13@150 13φ@150 ↓ D16@100 16φ@100	750×750 ↓ 800×800	12-D32 ↓ 16-D41	D13@150 13φ@150 ↓ D16@100 16φ@100	500×750 ↓ 600×1000	4-D25 2-D22 ↓ 4-D41 2-D38	4-D13@175 ↓ 4-D16@125	

The storey shear capacity of this building was designed based on the beam yielding mechanism, and was expected to exceed the lateral shear force corresponding to a base shear coefficient of 0.18. When the beams have all reached their yield points, the factored (x1.25) stress of the columns must not exceed the design range (Fig.3).

COLUMN TEST SERIES Column specimens were 1/2.6 scale models of 2nd floor exterior columns with additional core re-bars (Fig.1,2). Materials used are listed in Table 2.

To trace the actual stress path of the exterior columns during earthquake as closely as possible, cyclic shear and axial forces were applied simultaneously to the model columns. This loading pattern was decided according to the results of static frame analysis. Fig.3 shows the bending moment and axial force interaction curve of the column. The two solid lines show the loading passes of V-1 and V-2. In the final loading cycle, the models were loaded until the shear force exceeded the calculated flexual strength, under large constant axial force (V-1 was 60% of the tensile strength, V-2 was 70% of the compressive strength of the column).

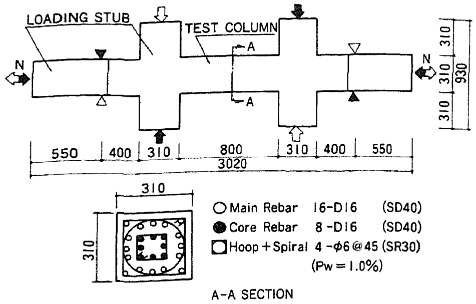


Fig.2 Column specimen

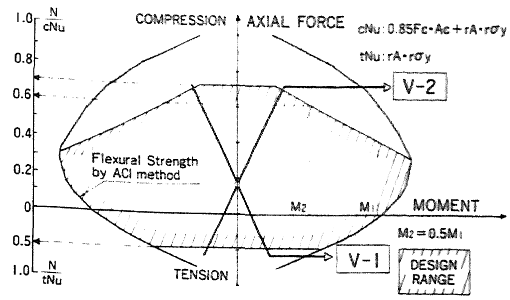


Fig.3 Moment-axial force interaction and loading pass

Table 2 Materials

Steel	Yield kgf/mm²	Ultimate kgf/mm²	El. %
D16	40.0	60.0	25
φ6	34.5	50.0	35

Concrete	Compressive Strength kgf/cm²	Tensile Strength kgf/cm²	Elastic Modulus kgf/cm²
Fc=420	435	38.4	257000

BEAM-COLUMN SUBASSEMBLAGE SERIES Specimens F-1 (exterior) and F-2 (interior) were half scale models of the 2nd floor frame (Fig.1,4). Specimen F-5 was a modification of F-1, and F-3 and F-4 were modifications of F-2. To study the shear strength of the joint core concrete, F-3,F-4 and F-5 had no shear reinforcement in the joint core concrete or transverse beams. Materials used in these specimens are listed in Table 3.

Column axial stresses were kept constant : F-2 and F-3 at 20% of concrete strength and F-1,F-4 and F-5 at 0%. Earthquake-type reversed cyclic loads were then applied at both tips of the beams, until the storey deformation angle (Rs) reached 1/20 rad.

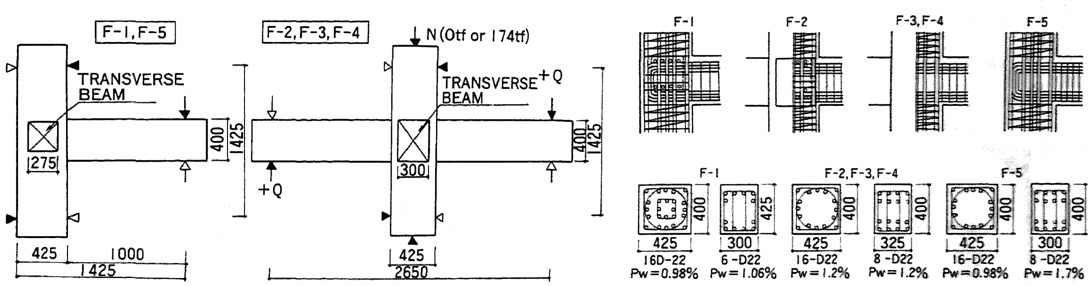


Fig.4 Beam-Column subassemblage specimen

Table 3 Materials

Steel	Yield kgf/mm²	Ultimate kgf/mm²	El. %
D22	40.0	60.1	25
φ9	35.2	49.8	34

Concrete	Compressive Strength kgf/cm²	Tensile Strength kgf/cm²	Elastic Modulus kgf/cm²
F-1,F-2	414	36.5	251000
F-3,F-4,F-5	454	35.3	271000

COLUMN TEST RESULT

Fig.5 shows the hysteresis loops of the exterior columns. The measured loads are listed in Table 4, together with some predicted values. Photo 2 shows post-test specimens.

The influence of axial force on the restoring force characteristics was significant. Increasing tensile force decreased cracking load and column stiffness. When axial force became compressive, no cracks were observed and the restoring force characteristics remained elastic.

Within the design range loadings, although minor cracking was observed in the tensile region, the columns suffered no serious damage and the hysteresis loops were stable (no strength or stiffness deterioration was observed).

In the final loading cycle, the deformation of column V-1 (115tf in tension) increased to Rs=1/25rad without major damage as the shear force was gradually increased. Model column V-2 (362tf in compression) behaved elastically until Rs reached about 1/400rad i.e. longitudinal reinforcement started to yield in compression. Then the sudden crushing of the concrete decreased column stiffness noticeably at Rs=1/200rad and the restoring force deteriorated as the deformation increased. The large axial force applied to the column, however, was supported stably until about Rs=1/50rad. Even though the columns were subjected to a very large axial force in the final cycle, the observed maximum shear force of each column exceeded the flexual strength caluculated by the ACI method (See Table 4).

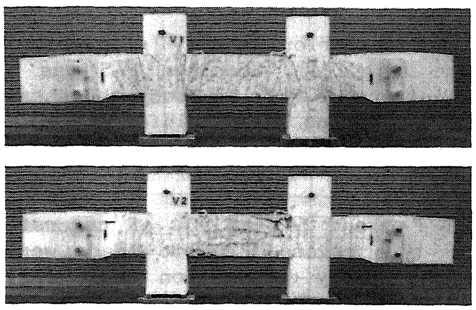
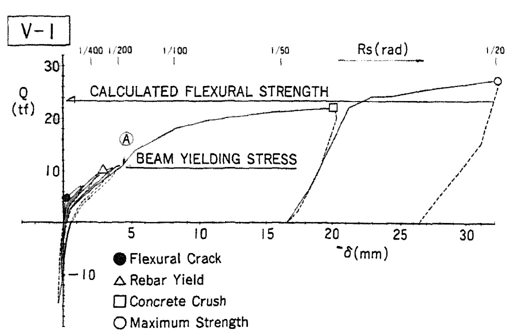


Photo.2 Post-test specimens

Table 4 Test results

	Fc	Sc	Cc	My	Cy	Qu	Qf	Qu/Qf
V-1	4.7	8.3	22	9.8	11	27.7	23.9	1.18
V-2	33	40	22	35	—	43.0	39.8	1.08

Note Columm Shear Force Q (tf)
Fc:Flexual Crack Sc:Shear Crack Cy:Core Rebar Yield Qu:Maximum Capacity
Cc:Concrete Crush My:Main Rebar Yield Qt:Calculated Flexual Strength (by ACI method)



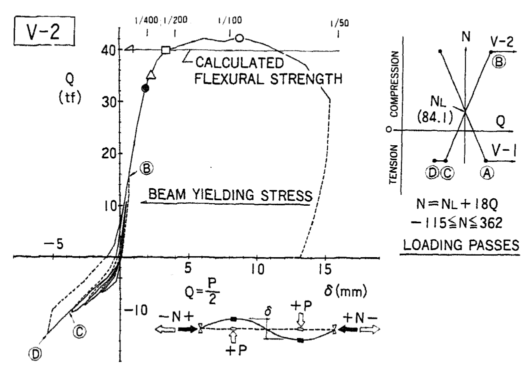


Fig.5 Hysteresis loops of columns

BEAM-COLUMN SUBASSEMBLAGE TEST RESULT

Fig.6 shows the load-displacement curves of F-2 and F-4. Fig.7 shows the envelope curves of interior models F-2,F-3 and F-4. Measured loads are listed in Table 5 together with some predicted values. Photo 3 shows the post-test specimens.

F-1 and F-2 showed typical restoring force characteristics of weak-beam and strong-column frames. Plastic hinges formed in the beam ends at about Rs=1/100rad. Significant stiffness degradation was observed at this point. However, the restoring force increased steadily as deformation increased and the hysteresis

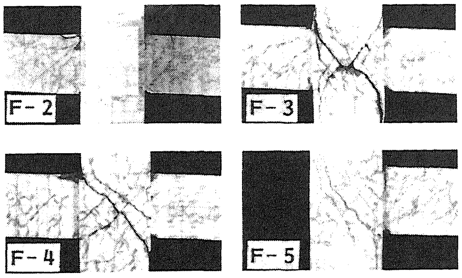


Photo.3 Post-test specimen

Table 5 Test results

	Fc	My	Cc	Psc	Pcc	Qu	Qf	Qu/Qf
F-1	3.5	25.8	26.7	—	—	30.9	25.4	1.22
F-2	3.0	31.5	30.2	—	—	34.0	31.1	1.09
F-3	6.5	30.1	30.6	11.6	26.3	31.4	31.1	1.00
F-4	3.0	29.7	30.0	5.2	—	30.0	31.1	0.96
F-5	4.5	32.8	31.4	12.2	—	34.8	32.9	1.06

Note Beam Shear Force Q (tf)
Fc :Flexual Crack My :Beam Rebar Yield Pcc:Joint Concrete Crush Qu :Maximum Capacity
Cc :Concrete Crush Psc:Joint Shear Crack Qf :Calculated Beam-Yielding Strength

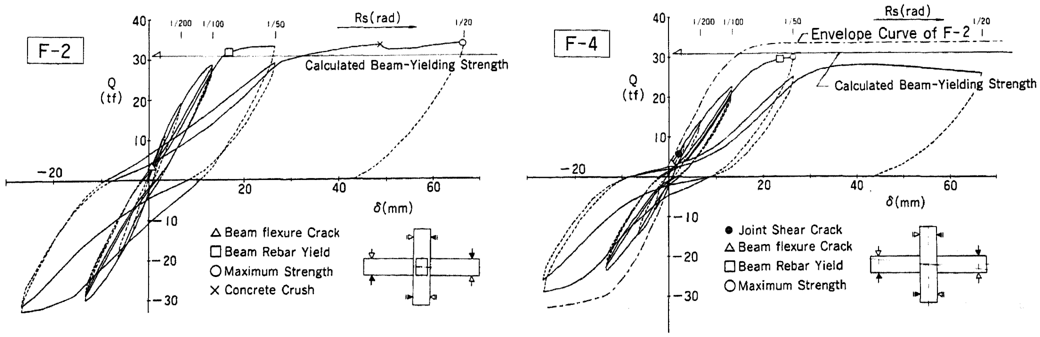


Fig.6 Hysteresis loops of beam-column subassemblages

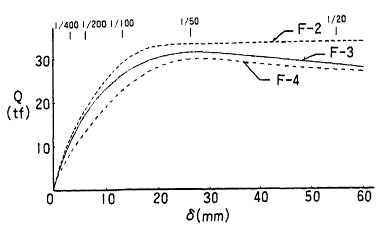


Fig.7 Envelope curves

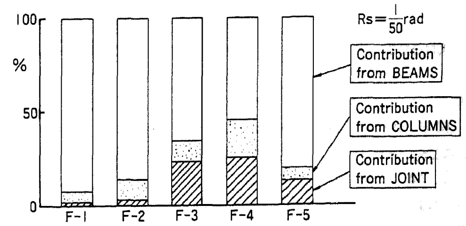


Fig.8 Relative contributions of each member

loops remained stable. The observed maximum force exceeded the value calculated by the beam yielding mechanism by 10 to 22%.

The hysteresis loops of F-3,F-4 and F-5 showed clear effects of shear failure of the joint core concrete. Many shear cracks occurred in the joint core concrete at relatively low loading and the cracks degraded specimen stiffness. The maximum restoring forces were observed at about Rs=1/50rad which was about twice as large as for F-1 and F-2. Then the restoring force decreased gradually and the hysteresis loops clearly changed to the slipping type. The observed maximum restoring forces varied around beam-yielding strength (from 97% to 106%).

Fig.8 shows the relative contributions of each part of the subassemblage (beam,column or joint) to total storey deformation at Rs=1/50 rad. The beam yielding specimen F-1 and F-2 showed a high contribution from beams (90 to 95%). Howevwre, for the shear failing specimen F-3,F-4 and F-5, 15% to 30% of the storey deformation was due to joint shear deformation and the contribution from beam deformation was 40 to 60%.

Observed shear cracking stresses and maximum shear stresses in the joint core concrete are listed in Table 6. The axial force in the column increased the shear

Table 6 Maximum and cracking shear stresses of the joint

	$_e\tau_c$	$_e\tau_u$	$_c\tau_u$
F-1	—	76	108
F-2	—	152	129
F-3	46	141	113
F-4	21	134	113
F-5	27	87	85

Note Joint Shear Stress τ(kgf/cm²)

$_e\tau_c$:shear Crack $_e\tau_u$:Maximum Shear
$_c\tau_u$:Calculated Shear Strength by ACI-352

$$\tau = \frac{\Sigma M_B}{\left(1+\frac{d_p}{h_o}\right)\ {}_eV_c},\quad {}_eV_c = j_b \times j_c \times t_p$$

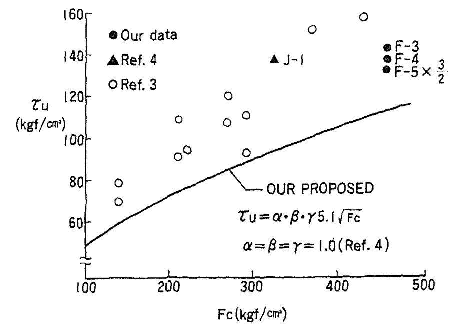


Fig.9 Shear strength of the joint core

cracking stress of the joint core concrete, but had little influence on its shear strength, as reported by previous research (Ref.2). Fig.9 shows the relation between shear strength of the joint core and concrete compressive strength. In this figure the observed maximum shear strength of F-3,F-4 and F-5 (multipled by 3/2, Ref.3) are plotted together with some other experimental data summarized by Ogura et al. (Ref.4) and our previous data (Ref.3).

The test result shows that shear strength of the joint core concrete increased with concrete compressive strength, and the empirical design formula is valid up to a concrete strength of 450kgf/cm2.

CONCLUSIONS

From these experimental investigations, the following conclusions can be derived.

1) Within the design range loading, exterior columns with additional core re-bars and high strength concrete suffer no serious damage. Thus the design range is verified to be adequate.
2) The axial force fluctuation significantly affects the restoring force characteristics of the column. This must be considered in the structual design of high-rise R/C buildings.
3) Under a large compressive and tensile force, the maximum strength of the column with core re-bars exceeds the value calculated by the ACI method.
4) The beam-column subassemblages using high strength concrete are verified to develop a beam-yielding mechanism. The restoring force characteristics of these frames are stable and have sufficient ductility (ductility factor is over 5).
5) Our proposed empirical design formula for the shear strength of the joint core concrete is verified to be valid up to a concrete strength of 450kgf/cm2.

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