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ON THE UNBONDED PRESTRESSED CONCRETE MEMBERS IN SEISMIC STRUCTURES

Minehiro NISHIYAMA[1], Hiroshi MUGURUMA[1] and Fumio WATANABE[2]

[1]Department of Architectural Engineering, Kyoto University, Sakyou-ku, Kyoto, Japan
[2]Department of Architecture, Kyoto University, Sakyou-ku, Kyoto, Japan

SUMMARY

The experiment was carried out on unbonded prestressed concrete beams and bonded beams. The test results showed that there was little difference in hysteretic restoring force characteristics between unbonded and bonded beams. The fluctuation of tendon stress measured at the anchorage end in bonded beams was 1.5 times larger than that in unbonded beams, because of bond deterioration in the anchorage region of beam-column joint. From another experiment which was carried out on reinforced concrete portal frame with an unbonded prestressed concrete beam, the tendon stress increment was so small even at large story drift angle that it may be unnecessary to consider any risk of tendon fracture. In addition, the analytical results of the portal frame, which was based on the stress-strain relation of materials, showed good agreement with the experimental results.

INTRODUCTION

In seismic area, the application of unbonded prestressed concrete to primarily earthquake resistant members is prohibited due to several reasons, i.e., safety of tendon anchorage assembly against cyclic earthquake load, uncertainties with regard to the fluctuation of tendon stress, little available data on hysteretic restoring force characteristics and complexity in analysis, etc. However, unbonded prestressed concrete members can be very useful to develop the further demand for prestressed concrete structure, because of economical advantage of unbonded tendon, that is, no need for grouting at the construction site, and of practically perfect protection against corrosion comparing with the grouting which is likely to be imperfect. In addition, the past researches reported that small amount of additional nonprestressed reinforcement can improve the restoring force characteristics and the flexural ductility [1,2].

The objects of this study are (1) to present the data on the behavior, especially on the fluctuation of tendon stress at anchorage end, of unbonded prestressed concrete beams and framed structures obtained by both experimental and analytical method, and (2) to prove the possibility of the use of unbonded prestressed concrete members as earthquake resisting members. The experiments are carried out on unbonded and bonded prestressed concrete beams, and on the reinforced concrete portal frame with an unbonded prestressed concrete beam under high-intensity reversed cyclic loading. The test results obtained from unbonded beams are compared with those from bonded beams in terms of restoring force characteristics and fluctuation of tendon stress at the anchorage end. Besides the experiments, the analytical results are presented, which obtained from the computer program to predict the behavior of unbonded prestressed concrete members. It is based on the stress-strain relation of the materials and bond-slip relation between concrete and tendon.

EXPERIMENT ON THE UNBONDED PRESTRESSED CONCRETE BEAMS

Test Specimens and Test Procedure

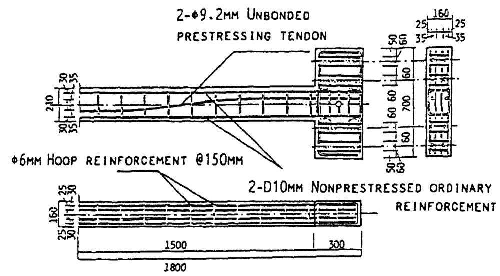

Fig.1 Dimensions and reinforcing details

Fig.1 shows the dimensions and reinforcing details of the specimen U35CR as a typical example. The specifications of the specimens are listed in Table 1. The following indication of specimen was used regarding to the test parameters.

U 35 C R

- R — Reversed cyclic loading
- C — Tendon profile (C:draped, S:straight)
- 35 — Eccentricity of tendon measured from the centroidal axis of the beam in mm
- U — U:unbonded beam, B:bonded beam

Table 1 Specifications of the specimens

Specimen	U35CR	U35SR	U60CR	U60SR	B35CR	B35SR	B60CR	B60SR
Prestressing Tendon	2-ϕ9.2 SBPR 110/125				2-D9.2 SBPD 110/125			
Eccentricity of Tendon in mm (e/D)	35 (1/6)		60 (1/3.5)		35 (1/6)		60 (1/3.5)	
Prestressing Tendon Index q_{pt}	0.17	0.17	0.14	0.14	0.21	0.21	0.18	0.18
q_{po}	0.34	0.33	0.53	0.51	0.41	0.41	0.64	0.64
Concrete	C1	C2	C1	C2	C3	C3	C3	C3
Effective Prestressing Force in kN	102.4	121.1	103.7	117.1	113.6	114.1	113.9	111.2
$\sigma pe/\sigma py$	0.59	0.72	0.60	0.69	0.67	0.67	0.67	0.67
$Pe/b \cdot D \cdot fc'$	0.07	0.08	0.07	0.08	0.10	0.10	0.10	0.10
Concentrated Load at midspan in kN	9.81	5.0	16.68	8.83	9.42	4.91	16.68	8.24

e : eccentricity of tendon measured from the centroidal axis of the beam section
D : total depth of beam section b: breadth of beam section
σpe : effective prestress in prestressing tendon
σpy : yield strength of prestressing tendon
Pe : effective prestressing force
fc' : compressive strength of concrete
$q_{pt} = Ap\ \sigma py/(b\ dp\ fc')$
$q_{po} = Ap\ \sigma py/(b(D-dp)fc')$
Ap : sectional area of prestressing tendon
dp : effective depth of tendon

Mechanical properties of ordinary reinforcements and prestressing tendon are summarized in Table 2. Those of concrete at the age of test are listed in Table 3.

Table 2 Mechanical properties of reinforcements

Reinforcements	D10	ϕ6	ϕ9.2 tendon
Yield stress in MPa	376	350*	1305*
Yield strain in %	0.204	-	-
Elastic modulus in 10^5 MPa	1.84	1.89	1.98

* 0.2% offset yield stress

Fig.2 shows the schematic figure of loading procedure. For applying reversed cyclic transversal load as simulated earthquake load, one end column stub was fixed while the other end was moved up- and downward without stub rotation. Thus, the beam was subjected to an anti-symmetric flexure with constant shear force over the whole length of the beam. In addition, the constant load as the live weight was applied by servo-actuator. Flexural moment applied to the beam ends were measured by load cells shown in Fig.2. Each end of these load cells had an universal joint so that only the axial force acted on them could be measured. Tensile force increments of pre-stressing tendon due to the load application were obtained by load cells inserted at the anchorage ends.

Table 3 Mechanical properties of concrete

Concrete	C1	C2	C3
Compressive strength in MPa	39.5	45.8	36.5
Tensile strength in MPa	3.27	4.21	2.97
Initial tangent modulus in 10^4 MPa	3.91	2.97	2.28
Strain at fc' in %	0.18	0.24	0.23

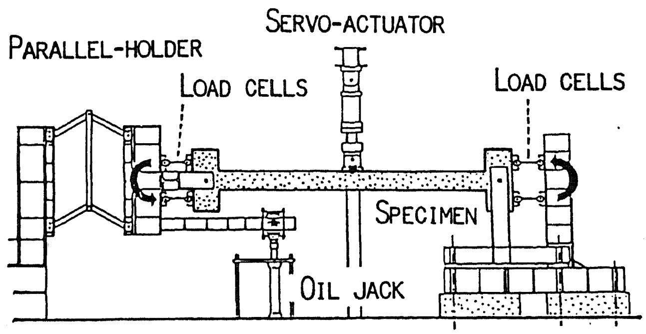

Fig.2 Loading setup

Test Results All specimens failed in flexure with crush of concrete and buckling of ordinary reinforcing bars in compression zone. In Fig.3a and 3b, shear force versus deflection relation of U60CR and B60CR are illustrated as a typical example. Here, shear force was defined as the sum of flexural moments at both beam ends divided by the beam length. Although each loading cycle consisted of four cycles in U60CR or five cycles in B60CR, only the first cycle in each series of deflection cycles is presented in those figures. From the figures, it can be observed that there was little difference in hysteretic restoring force characteristics between unbonded and bonded beam. The same tendency can be observed in the remainder of the specimens; U35CR versus B35CR, U60SR versus B60SR and U35SR versus B35SR.

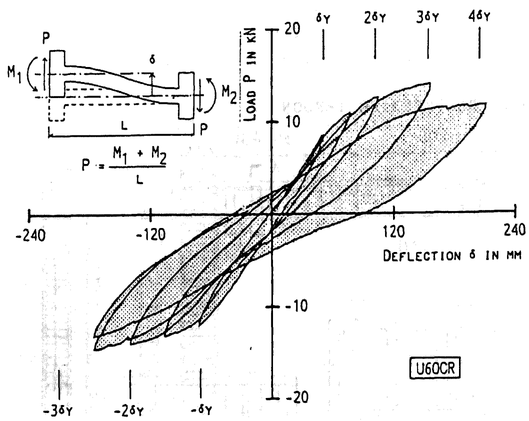

Fig.3a Typical shear force - deflection relation

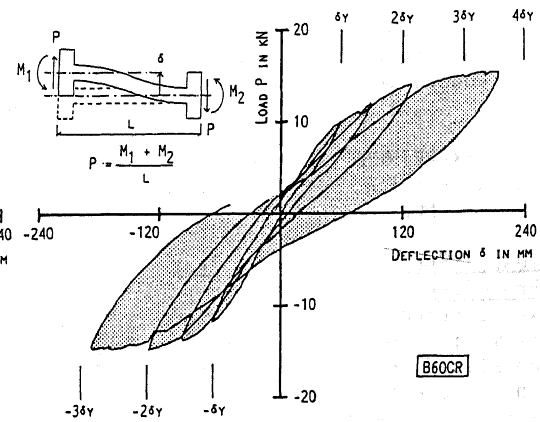

Fig.3b Typical shear force - deflection relation

Fig.4a and 4b show the fluctuation of tendon stress measured at the anchorage end of the beam in U60CR and B60CR. Up to the present it has been thought that the fluctuation of tendon stress at the anchorage end of unbonded beams was larger

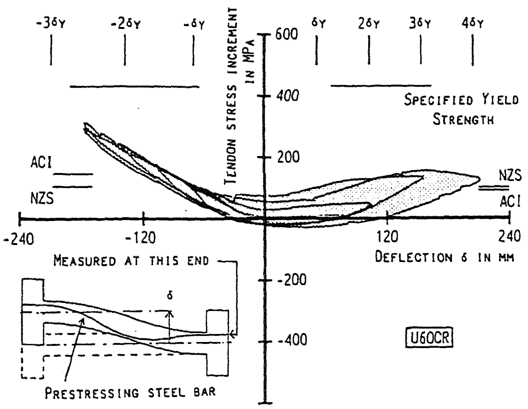

Fig.4a Typical tendon stress increment - deflection relation

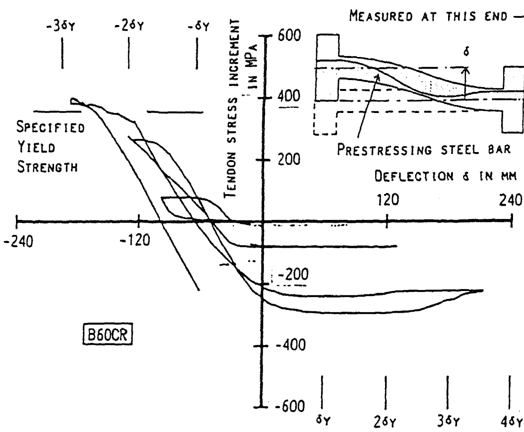

Fig.4b Typical tendon stress increment - deflection relation

than that of bonded beams and there might be some fear of the fracture of the unbonded tendon under reversed cyclic loading like earthquake load. However, as shown in the figures, the increment of tendon stress of the bonded beam B60CR was 1.5 times larger than that of U60CR. The reason for this is that the expected bond action between concrete and prestressing tendon in the column stub deteriorated easily under high-intensity reversed cyclic loading, and most of the increment of tendon force at the critical section was transferred to the anchorage end. Therefore, so far as this test is concerned, it is not proper that the unbonded member has a larger possibility of low-cycle fatigue failure at the anchorage end than the bonded member.

EXPERIMENT ON THE UNBONDED PRESTRESSED CONCRETE PORTAL FRAMES

Test Specimens and Test Procedure The experiment was carried out on two portal frames; One frame, 'FR35', consisted of the beam where the eccentricity of prestressing tendon was 35 mm and the other, 'FR60', has the prestressing tendon whose eccentricity was 60 mm. Fig.5 shows the dimensions and reinforcing details of specimen. These two frames were so designed as to have the same lateral load resistance. The moment capacity of the column was about 1.5 times that of the beam, so that the plastic hinges were intended to be located in beam ends and column bases. The mechanical proper-

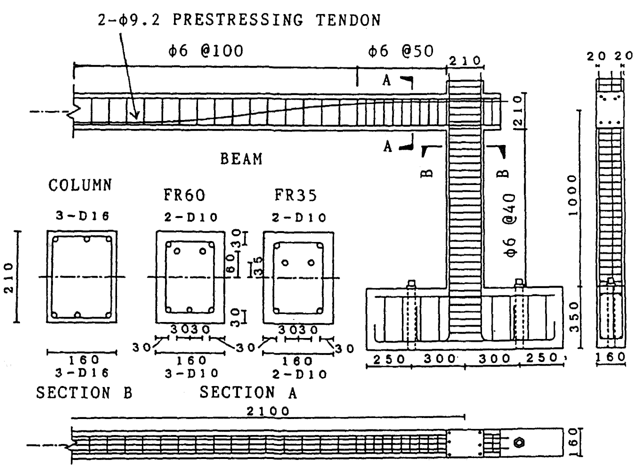

Fig.5 Dimension and reinforcing details

Table 4 Mechanical properties of concrete

Specimen	FR35	FR60
Compressive Strength fc' in MPa	36.9	36.8
Strain at fc' in %	0.244	0.218
Tensile Strength ft in MPa	3.4	3.3

Table 5 Mechanical properties of reinforcements

Reinforcement	D10	D16	ϕ6
Yield Strength fy in MPa	350.6	346.3	395.9
Yield Strain ε_y in %	0.205	0.180	0.224
Modulus of Elasticity in 10^5 MPa	1.72	1.92	1.76

ties of concrete and reinforcements are listed in Table 4 and 5. Specified 0.2% offset yield strength and tensile strength of prestressing tendon were 1078 MPa and 1225 MPa, respectively. Effective prestress were 107.8 kN for FR35 and 63.7 kN for FR60.

Reversed cyclic horizontal load was statically applied to the midspan of the beam by hydraulic jack. Besides the horizontal load, the vertical load was also applied at the midspan, so that the bending moments at the beam ends and the midspan due to the prestressing were offset at the beginning of the test. This vertical load was kept constant during the test. The "first yield" displacement of the frames were found when all the tension reinforcements in expected hinging regions had yielded. The first loading cycle consisting of ten cycles was followed by a series of deflection controlled cycles in the inelastic range, also comprising ten full cycles to each of the displacement ductility factors of ±2, ±3, ±4, and sometimes higher.

<u>Experimental Results and Comparison with the Analytical Results</u> Figs.6a and 6b show the first cycle in each series of deflection cycles of the measured horizontal load versus horizontal deflection at the midspan of the beam. These figures also show the calculated load-deflection curves. The calculation was done by 'Layer Element Method'. Further information is given in reference 3. Each loading cycle comprised only one full cycle, because the deterioration due to load cycles in concrete and bond was not considered in this calculation. When the experimental results were compared with analytical results, fairly good agreement could be observed. However, the larger the deformation became, the larger difference could be observed. It is mainly because the shear deformation was not taken into consider in the calculation. The shear deformation, especially in the column, dominated the whole deformation of the frame in the loading cycles to high ductility values, and some pinching of the load-deflection loops was noticeable in the experiment. Just before the failure, where the ductility factor was almost ±4, the shear deformation of the column base occupied a large portion of the whole deformation, while the deformation of

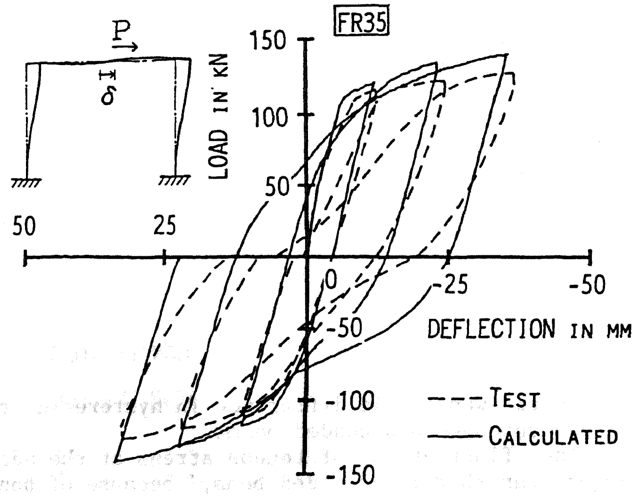

Fig.6a Lateral load - deflection relation in FR35

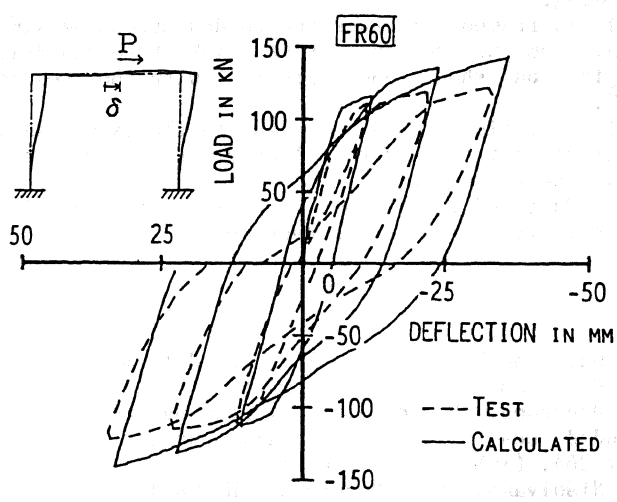

Fig.6b Lateral load - deflection relation in FR60

the beam remained small.

In Fig.7, the first cycle in each series of deflection cycles of the typical measured tendon force increment versus horizontal deflection at the midspan of the beam are shown. The calculated result is also shown in the same figure. As described before, the shear deformation at the column bases resulted in imposing not so large rotation on the beam ends even at large story drift. Therefore, the calculated results are larger than the experimental results. In addition, in the calculation, the tendon force increment continued to increase almost linearly with the deflection of the frame because the rotations at the beam ends had a linear relationship with the lateral deflection of the frame. The maximum tendon stress increment measured in the test was up to 196 MPa. From analytical and experimental results on the portal frame, the tendon force measured at the anchorage ends was not so large. It may be not necessary to consider any risk of tendon fracture even at large story drift. The tendon force increment measured in the test showed good agreement with the predicted value obtained from ACI and NZS within the available deflection.

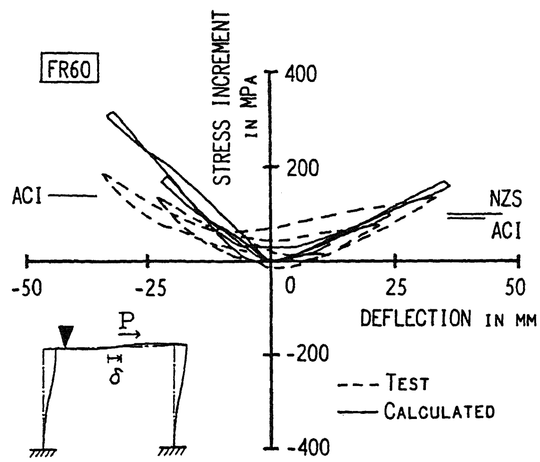

Fig.7 Typical tendon stress increment at the anchorage end - deflection relation

CONCLUSIONS

(1) There was little difference in hysteretic restoring force characteristics between unbonded and bonded beams.
(2) The fluctuation of tendon stress at the anchorage end in bonded beam was larger than that in unbonded beam, because of bond deterioration in the anchorage region of beam-column joint. Thus, it may be groundless to point out that the anchorage of unbonded tendon only involves the risk of failure under seismic loading.
(3) In framed structure the tendon stress measured at the anchorage ends was so small as not to consider any risk of tendon fracture even at large story drift angle and showed good agreement with the predicted value obtained from ACI and NZS.
(4) Analytical study was close to the experimental results on the restoring force characteristics and the fluctuation of the tendon stress of unbonded prestressed concrete portal frame.
(5) Unbonded prestressed concrete members could be utilized as structural components, even in seismic structures.

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