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INELASTIC SEISMIC BEHAVIORS OF CONCRETE BEAMS WITH HIGH-STRENGTH DEFORMED BARS

Kazuo KOBAYASHI and Susumu INOUE

Department of Civil Engineering, Kyoto University,
Yoshida-Honmachi, Sakyo-ku, Kyoto, Japan

SUMMARY

Reinforced concrete (RC) beams with high-strength deformed bar of nominal yield strength fsy=50kgf/mm^2 were tested under three types of loading. And, their inelastic load carrying behaviors after steel yielding were studied in comparison with those of RC beams with normal-strength one of fsy=30kgf/mm^2 in order to examine an applicability of high-strength bar for concrete structures.

INTRODUCTION

For a given ultimate flexural strength, it becomes possible to reduce a total area of reinforcing steel by using high-strength bars as tensile reinforcement of concrete members. This leads to cost down and simplicity of the construction. On the other hand, both of flexural crack width and deflection of the members under service load become larger because steel stress increase with the reduction in reinforcement ratio. From a view point of serviceability limit state, therefore, it may be necessary to introduce an adequate amount of prestress [Ref.1]. However, it is considered that high-strength steel can be applied effectively to the members, for instance, bridge pier in which axial compressive stress is relatively large under service load.

Some countries have a design code for the use of high-strength steel in concrete structures. Up to the present, Concrete Standard Code of Japan Society of Civil Engineers (JSCE) has not specified the design details for deformed bar having yield strength of more than fsy=50kgf/mm^2 (491MPa), although Japanese Industrial Standard (JIS) classifies the high-strength deformed bar of fsy=50kgf/mm^2 (491MPa) as SD50. This is mainly due to that sufficient fundamental data are not available as for the inelastic seismic behaviors of concrete structures reinforced with such high-strength steel.

The main object of this study is to make clear the load carrying capacity and inelastic deformation properties after steel yielding of RC beams reinforced with high-strength deformed bar in comparison with normal-strength one, and then to investigate the applicability of high-strength steel for reinforcement of the concrete structures or members.

EXPERIMENTAL PROGRAMS

<u>Reinforcing Bars</u> High-strength steel - ribbed type deformed bar having nominal

yield strength of fsy=50kgf/mm^2 (491MPa) (SD50) - was used for main reinforcement of tested beams in comparison with normal one of fsy=30kgf/mm^2 (294MPa) (SD30). The results of their tensile tests are indicated in Table 1.

Table 1 Results of Tensile Tests of Bars

Kinds of Bars	Yield Strength (kgf/cm^2)	Tensile Strength (kgf/cm^2)
SD30 D13	3640	5600
SD30 D16	3820	5920
SD50 D13	5370	6940

conversion factor:
1 kgf/cm^2 = 0.0981 MPa

<u>Tested Beams</u> Test programs were divided into three series according to the loading types, that is, i) uni-directional loading (Series-A), ii) reversed cyclic loading without load repetitions at gradually increased deflection amplitudes (Series-B) and iii) reversed cyclic loading with load repetitions at given deflection amplitudes (Series-C). Tested beams had a rectangular cross section of width × full depth = 10 × 20 cm and length of 160 cm as shown in Fig.1. Type-1 of Series-A beams were singly reinforced, while Type-2 of Series-A beams, Series-B and Series-C beams were doubly reinforced with equal amount of tension and compression steels.

The design compressive strength of concrete was chosen as 450kgf/cm^2 (44MPa) for Type-1 of Series-A beams and 400kgf/cm^2 (39MPa) for other beams. All the tested beams were so designed as to have almost the same ultimate flexural strength, that is, almost the same reinforcing steel index (q=Asfsy/bdfc') among the same series.

For the beams of lateral confinement type, the closed ties of ϕ6mm (fsy=49kgf/mm^2 (481MPa)) with 135° hook were arranged at the spacing of d/2 or d/4 (d:effective depth) within an expected yield hinge region of 60cm length over a mid span. In addition, the rectangular stirrups of ϕ6mm were also provided as web reinforcement at the spacing of 10cm in the remaining parts of the span in accordance with ACI 318-83 [Ref.2].

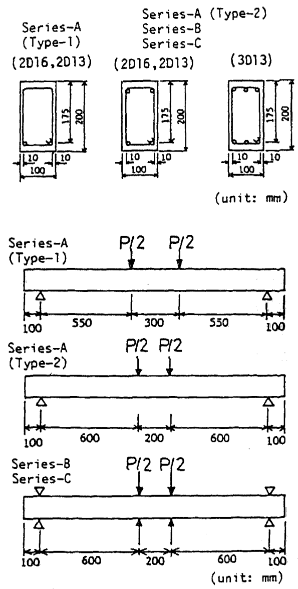


Fig.1 Cross Section of Tested Beams and Loading Patterns

<u>Loading Tests</u> The details of each loading type are as follows.

(1) Series-A: Series-A beams were divided into two types (Type-1 and Type-2). The former had a shear span length of a=55cm (a/d=3.14) and the latter a=60cm (a/d=3.43) as shown in Fig.1. All the Series-A beams were cyclically loaded up to failure under symmetrical two-point loads in one direction, each being fully unloaded when the deflection at loading points (δ) reached 3mm (θ=0.005), 6mm (θ=0.01), 9mm (θ=0.015), ----. The rotation angle, θ, was defined by the ratio of δ/a.

(2) Series-B: Series-B beams were subjected to gradually increased reversed cyclic loading with each one cycle, in which the deflection amplitudes were kept equal in the positive and negative directions. Each deflection amplitude was set to be δ= ±9mm (θ=±0.015), ±18mm (θ=±0.03), ±27mm (θ=±0.045), ±36mm (θ=±0.06) and ±45mm (θ= ±0.075).

(3) Series-C: Six beams of Series-C (Type-3) underwent 30 cycles of load reversal at a constant deflection amplitude of δ=±9mm (θ=±0.015) which is well in excess of steel yielding, and thereafter load repetitions were applied up to failure at δ= ±27mm (θ=±0.045). The θ-value of 0.015 corresponds to the maximum allowable design story drift of 1.5% suggested by Applied Technology Council Recommendations [Ref.3] for earthquake resistant reinforced concrete frame buildings. Remaining four beams (Type-4) were subjected to 10 cycles of load reversal at δ=±18mm (θ= ±0.03) during the first stage, and then each one cycle at δ=±27, ±36 and ±45mm up to failure.

The deflection at the first yielding of tensile steel (δy) was approximately 6mm (θy=0.01) for all the tested beams.

RESULTS OF TESTS AND DISCUSSIONS

Series-A Tests Table 2 indicates the measured yield load (Py) and ultimate flexural load (Pu) of Series-A beams. The calculated ultimate flexural load (Pu') was estimated by the conventional ultimate strength theory, assuming a rectangular stress block equal to 0.85 times the measured compressive strength of concrete and using an actual yield strength of reinforcing bar.

All the Series-A beams failed finally in flexure. In both case of Type-1 and Type-2, the Pu/Py ratio of the beam reinforced with high-strength bars is found to be somewhat smaller than that of the corresponding beam with normal-strength bars.

The measured ultimate load of the beams with high-strength bars, as well as the beams with normal-strength ones, is larger by at least 10% in Type-1 and 40% in Type-2 than the calculated one. Therefore, the ultimate flexural strength of beam reinforced with high-strength bars can be considered to be safely estimated by applying the conventional ultimate strength theory.

Table 2 Details of Series-A Beams

Specimen		Used Bars	*1 As	*2 s	*3 q	*4 Py (tonf)	Ultimate Load *5 mea. Pu (tonf)	Ultimate Load *5 cal. Pu' (tonf)	Pu/Pu'	Pu/Py
Type-1	AN-1	SD30	2-D16	∞	0.145	9.23	10.52	8.87	1.19	1.14
	AN-2	SD30	2-D16	d/2	0.147	9.00	9.90	8.87	1.12	1.10
	AN-3	SD30	2-D16	d/4	0.147	9.01	10.67	8.85	1.21	1.18
	AH-1	SD50	2-D13	∞	0.130	8.53	8.96	7.99	1.12	1.05
	AH-2	SD50	2-D13	d/2	0.130	8.46	9.14	7.99	1.14	1.08
	AH-3	SD50	2-D13	d/4	0.130	8.53	9.30	7.99	1.16	1.09
Type-2	AN-4	SD30	2-D16	∞	0.206	9.02	10.10	7.00	1.44	1.12
	AN-5	SD30	2-D16	d/4	0.206	8.90	11.00	7.00	1.57	1.24
	AN-6	SD30	3-D13	d/4	0.175	8.64	10.75	6.52	1.65	1.24
	AH-4	SD50	2-D13	∞	0.172	8.41	9.00	6.42	1.40	1.07
	AH-5	SD50	2-D13	d/4	0.172	8.10	9.40	6.42	1.46	1.16

*1: Type-1 beams were provided with tensile steel only.
Type-2 beams had an equal amount of compression steel.
*2: spacing of closed ties (∞: no closed ties)
*3: reinforcing steel index (q=Asfsy/(bdfc'))
*4: measured yield load at the first yielding of tensile steel
*5: Pu' was calculated ultimate flexural load estimated by ACI 318-83.
conversion factor:
1 tonf = 9.807 kN

Fig.2 shows the envelope curves of the measured load-deflection hysteresis loops together with calculated ones. The calculated curves were obtained by modifying the stress-strain relations of concrete and reinforcing steel proposed by Park et al. [Ref.4], and the index "1" and "2" of calculated curves correspond to ones in which the stress-strain relation of reinforcing steel in the strain hardening region is

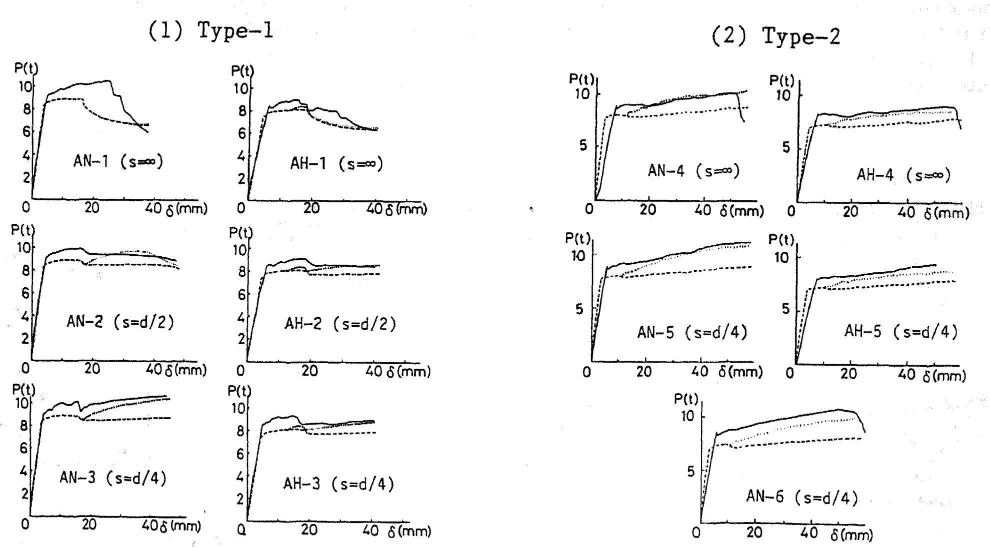


Fig.2 Load-Deflection Envelope Curves (Series-A)

·········Calculated Curve 1
------Calculated Curve 2

assumed to be non-linear and linear, respectively.

It can be clearly seen in Fig.2 that in the case of singly reinforced beams (Type-1) without lateral confinement, the load carrying capacity commences to decrease rapidly immediately after an attainable maximum load irrespective of steel types. This reduction, however, can be mitigated significantly by arranging an adequate amount of compression steel as done for Type-2 beam or closed ties.

Series-B Tests   Table 3 shows the details of Series-B beams and test results. Fig.3 shows the load-deflection hysteresis loops of Series-B beams together with an envelope curve of each corresponding Series-A beam.

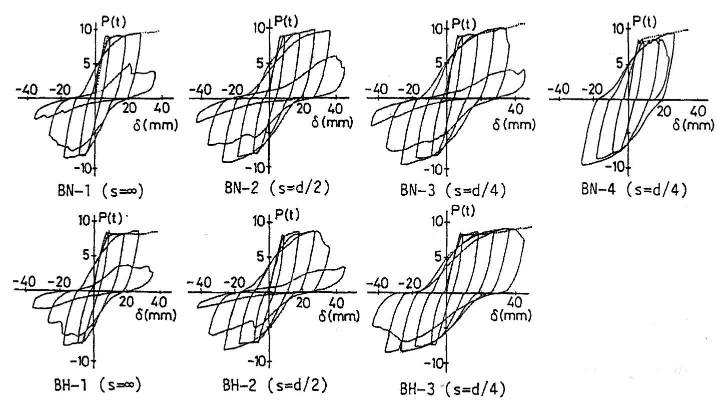
Table 3 Details of Series-B Beams

Specimen	Used Bars	As (=As')	s	q	measured Pu positive (tonf)	measured Pu negative (tonf)	calculated Pu' (tonf)
BN-1	SD30	2-D16	∞	0.206	9.38	8.60	7.00
BN-2	SD30	2-D16	d/2	0.206	9.64	9.39	7.00
BN-3	SD30	2-D16	d/4	0.206	10.18	9.51	7.00
BN-4	SD30	3-D13	d/4	0.175	9.48	9.64	6.52
BH-1	SD50	2-D13	∞	0.183	8.63	7.79	6.37
BH-2	SD50	2-D13	d/2	0.183	8.75	8.31	6.37
BH-3	SD50	2-D13	d/4	0.183	9.13	8.60	6.37

conversion factor:
1 tonf = 9.807 kN

It is clearly indicated in Fig.3 that the load carrying capacity of Series-B beam subjected to reversed cyclic loading commences to decrease at smaller deflection amplitude than that of each corresponding Series-A beam irrespective of steel types. This is due to that the X-shaped diagonal shear cracks extended remarkably at $\delta=\pm 18\sim\pm 21$mm ($\theta=\pm 0.03\sim\pm 0.045$) in the former beam. This implies that shear resistance carried by concrete deteriorates significantly under reversed cyclic loading in the post-yielding stage. The ductility, however, is found to be improved efficiently even in Series-B beams when provided with an adequate amount of lateral confinement, because a part of such X-shaped shear cracks crossed the closely spaced ties arranged over 20cm length of the shear span.

Fig.4 shows the equivalent coefficients of damping, heq, which are estimated from the load-deflection hysteresis loops of Series-B beams. The heq value of the beam reinforced with high-strength steel is somewhat smaller within the range of $\delta\leqq 27$mm ($\theta\leqq 0.045$) than that of the corresponding beam with normal-strength one. On the other hand, the deflection amplitude at which the heq value commences to decrease abruptly tends to be larger with increasing an amount of closed ties, because the ties arranged in the tested beams acted not only as lateral confinement but also as shear reinforcement. As far as Series-B tests are concerned, the heq value of the beams reinforced with high-strength deformed bars (reinforcing steel index of q=0.183) is approximately 0.15 and 0.27 at the deflection amplitudes of $\theta=\pm 0.015$ and ± 0.03 respectively, irrespective of tie spacing.



(……… Envelope Curves of Corresponding Series-A Beams)

Fig.3 Load-Deflection Hysteresis Loops (Series-B)

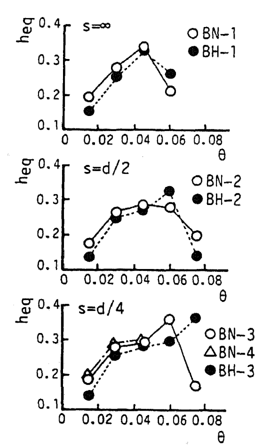


Fig.4
Equivalent Coefficient of Damping (Series-B)

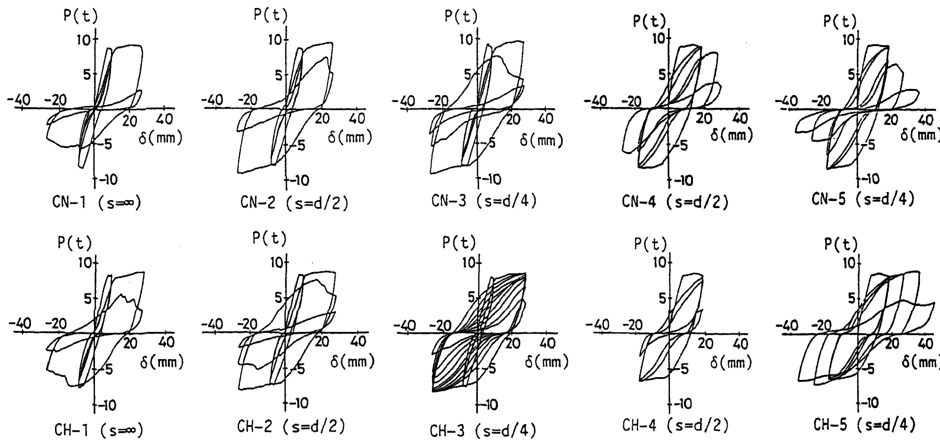


Fig.5 Load-Deflection Hysteresis Loops (Series-C)

Table 4 Details of Series-C Beams

Specimen		Used Bars	As (=As')	s	q	measured Pu		calculated Pu' (tonf)
						positive (tonf)	negative (tonf)	
Type-3	CN-1	SD30	2-D16	∞	0.206	9.26	8.46	7.00
	CN-2	SD30	2-D16	d/2	0.206	9.59	9.24	7.00
	CN-3	SD30	2-D16	d/4	0.206	9.76	9.48	7.00
	CH-1	SD50	2-D13	∞	0.183	8.74	7.50	6.37
	CH-2	SD50	2-D13	d/2	0.183	8.60	8.28	6.37
	CH-3	SD50	2-D13	d/4	0.183	8.63	8.50	6.37
Type-4	CN-4	SD30	2-D16	∞	0.206	9.13	8.49	7.00
	CN-5	SD30	2-D16	d/4	0.206	9.24	8.79	7.00
	CH-4	SD50	2-D13	∞	0.183	8.54	6.71	6.37
	CH-5	SD50	2-D13	d/4	0.183	8.96	7.38	6.37

conversion factor:
1 tonf = 9.807 kN

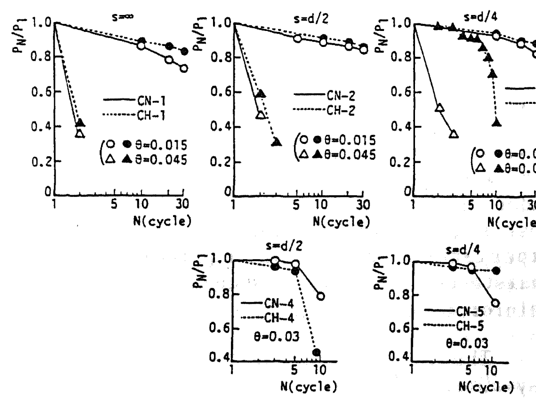


Fig.6 Strength Degradation with Repeated Cycles

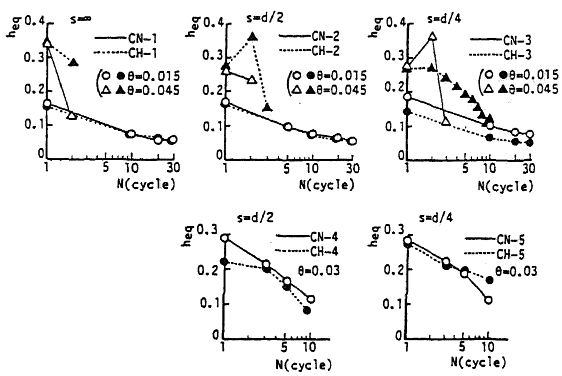


Fig.7 Changes of heq with Repeated Cycles

<u>Series-C</u> <u>Tests</u> Table 4 lists the details of Series-C beams and test results. Typical load-deflection hysteresis loops are shown in Fig.5, and the strength degradation with repeated cycles at a constant deflection amplitude are shown in Fig.6.

It is observed from Fig.5 and Fig.6 that the load-deflection hysteresis loops of Type-3 beams, irrespective of steel types, are clearly stable within N≈30 cycles of load reversals at $\delta=\pm9$mm($\theta=\pm0.015$) and the strength degradation after 30 cycles of load reversals is only approximately 20%, but that the load carrying capacity deteriorates abruptly at $\delta=\pm27$mm ($\theta=\pm0.045$) due to significant development of X-shaped shear cracks. In Type-4 beams, on the other hand, the hysteresis loops change gradually from stable to degraded shape even within N≦10 cycles at $\delta = \pm 18$mm ($\theta = \pm 0.03$) due to the same reason as in Type-3. However, such deterioration of load carrying capacity can be mitigated by arranging tie reinforcement closely, for example, at spacing of s=d/4.

Fig.7 shows the change of equivalent coefficient of damping (heq) with repeated cycles. In Type-3, the value of heq is approximately 0.15 and 0.30 at the first cycle of $\theta = \pm 0.015$ and ± 0.045, respectively. This value, however, decreases with increasing repeated cycles by pinching effect and reduces to less than 50% of that at the first cycle after 30 cycles at $\theta = \pm 0.015$. This tendency is observed more remarkably at the deflection amplitude of $\theta = \pm 0.045$. In Type-4, on the other hand, the heq-value at the first cycle of $\theta = \pm 0.03$ is approximately 0.27, but decreases to 40% of that after 10 cycles. As far as Series-C tests are concerned, heq-value is found to be not influenced remarkably by both of steel strength and tie reinforcement but affected considerably by deflection amplitude.

CONCLUSION

Inelastic behaviors of concrete beams reinforced with high-strength deformed bars (SD50: fsy=50kgf/mm^2 (491MPa)) were tested in comparison with those of RC beams with normal-strength ones (SD30: fsy=30kgf/mm^2 (294MPa)).

As far as these tests are concerned, significant differences could not be seen in the load carrying capacity and the inelastic deformation properties between these two types of RC beams when reinforced at an equal reinforcing steel index. Beam ductility could be improved remarkably by arranging an adequate amount of tie reinforcement. However, it should be noted that all the tested beams subjected to reversed cyclic loading failed finally in shear even when provided with web reinforcement according to ACI 318-83, while the beams failed in flexure under uni-directional loading. This implies that shear resistance of concrete is lost significantly under reversed cyclic loading in the post-yielding range as experienced during earthquakes. Therefore, it is suggested that the shear resistance of concrete should not be taken into account in the seismic design of reinforced concrete structures.

The results of these tests indicate that high-strength deformed bar of fsy=50kgf/mm^2 (491MPa), as well as normal-strengrh deformed bar of fsy=30kgf/mm^2 (294MPa), can be used well for reinforcing steel of concrete members subjected to earthquake action, although the tension steel stress should be limitted to an appropriate value when the control of crack width and deflection under service load is especially important in the former.

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