INELASTIC PERFORMANCE OF PRESTRESSED CONCRETE BEAMS UNDER CYCLIC FLEXURE

K. Okada (I)
K. Kobayashi (II)
S. Inoue (III)
Presenting Author: K. Kobayashi

SUMMARY

Totally thirty-four grouted and ungrouted post-tensioned beams were tested under two patterns of high intensity cyclic flexure with or without load reversals to study their inelastic performance such as load carrying capacity, elastic recovery, energy dissipation and ductility. The results of tests showed that the prestressing steel index, spacing of closed stirrup ties for lateral confinement and concrete strength had the considerable effects on those properties.

INTRODUCTION

Two types of post-tensioned beams, with bonded or unbonded prestressing bars, were tested under gradually increased cyclic loadings well into the post-elastic range. And, the effects of prestressing steel index, degree of lateral confinement and concrete strength on the fundamental inelastic behaviors of those beams were studied for assessing their ability to withstand seismic loading.

TEST PROGRAMS

Tested Beams

All of the post-tensioned beams tested were of a rectangular section of width (b) x full depth (h) = 100 x 200 mm and length of 1600 mm as shown in Fig. 1.

Test programs consisted of Series-A and Series-B, and the variables were chosen as follows.

1) Series-A Tests
   a) prestressing steel index, $q = pf_{py}/fc'$, varying from 0.081 to 0.260, in which p, f_{py} and fc' represent prestressing steel ratio (Ap/bd), actual yield strength of prestressing steel and compressive strength of concrete.
   b) two kinds of spacing of closed stirrup ties, for lateral confinement in seismic design, of s = 100 mm or d/4 (d: effective depth). These were selected on the basis of, for instance, New Zealand Code [1], and d/4 (d = 160 mm) was smaller than 100 mm in all of Series-A beams.

(I) Prof. of Civ. Engrg., Kyoto Univ., Kyoto, Japan
(II) Assoc. Prof. of Civ. Engrg., Kyoto Univ., Kyoto, Japan
(III) Graduate Student of Civ. Engrg., Kyoto Univ., Kyoto, Japan

c) two kinds of design strength of concrete of $fc' = 500\ kgf/cm^2$ (W/C = 42%) and $800\ kgf/cm^2$ (W/C = 31%). The latter high strength concrete was made by using a high-range water-reducing admixture.

2) Series-B Tests

a) q: three levels of 0.133, 0.216 and 0.241

b) s: three levels of ∞, d/2 and d/4, in which d was 140 mm in all of Series-B beams.

c) bonded and unbonded

Concrete of $fc' = 450\ kgf/cm^2$ (W/C = 44%) was used in all of Series-B beams.

Fabrication of Beams

The ø6mm closed stirrup ties of $fsy = 35\ kgf/mm^2$ were arranged at the prescribed spacings within 500 mm length over a mid-span, which covered the plastic hinge zone. And, in order to prevent the premature shear failure, web reinforcement of ø6mm vertical stirrups were provided in the parts other than 500 mm length over the mid-span. The amount of web reinforcement was determined on the basis of the JSCE PC Code[2], in which the shear carried by concrete (Vc) was taken into account in contrast to the New Zealand Code[1] and FIP Recommendation[3] which assume Vc as zero for reversed cyclic loading because of reduction in the effectiveness of the concrete shear resistance mechanism.

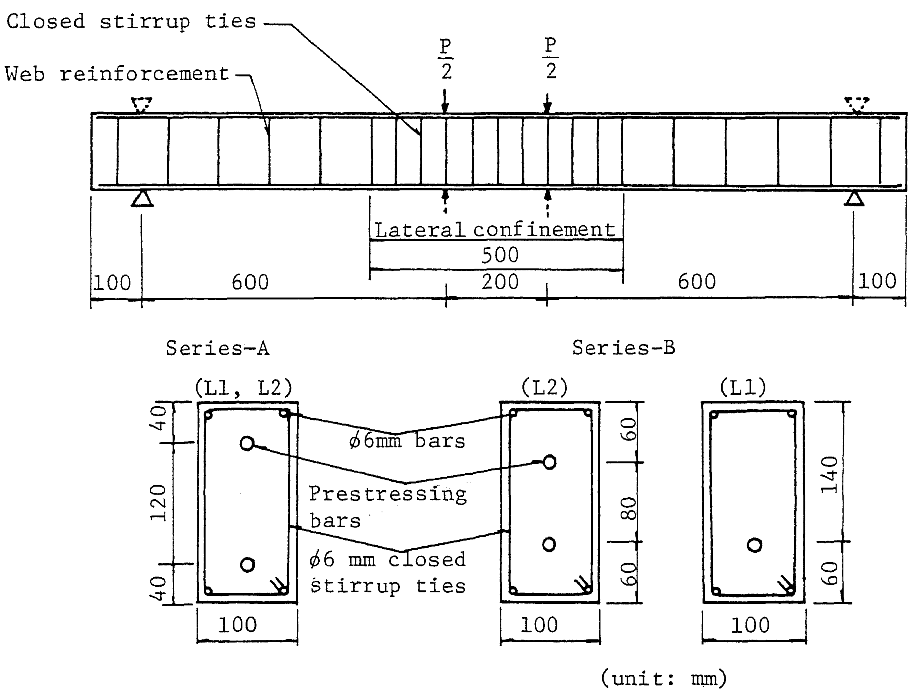

Fig. 1 Details of Tested Beams

Table 1. Kinds of Beams and Results of Tests

Beams		Loading Type *1)	Grout *2)	fc' *3)	pc Bars *4)	q	s (mm) *5)	σp (kgf/cm^2) *6)	Mea. Flexural Cracking Loads (t) *7)	Max. Ult. Loads, Pmax (t) Mea.	Cal.
Series-A	A1				Cφ11	0.124	100	62	3.75	8.50	6.93
	A2						d/4	62	4.00	8.75	6.93
	A3				Cφ13	0.169	100	108	6.50	10.75	9.24
	A4			500			d/4	106	6.00	10.85	9.24
	A5				Cφ15	0.196	100	147	7.50	12.45	10.60
	A6	L1					d/4	147	7.50	12.10	10.60
	A7				Dφ9.2	0.081	d/4	68	5.00	7.85	4.62
	A8				Cφ17	0.208	100	148	7.50	14.10	12.80
	A9			800			d/4	152	8.00	14.40	12.80
	A10				Cφ19	0.260	100	185	9.00	14.85	15.50
	A11		B				d/4	195	9.00	15.45	15.50
	A12				Cφ11	0.138	100	61	4.00 (4.00)	8.60	6.87
	A13						d/4	58	4.00 (4.00)	8.05	6.87
	A14				Cφ13	0.189	100	102	6.00 (6.00)	10.25	9.14
	A15			500			d/4	118	6.00 (6.00)	10.25	9.14
	A16	L2			Cφ15	0.219	100	146	6.00 (7.00)	10.75	10.40
	A17						d/4	151	7.00 (7.00)	11.05	10.40
	A18				Dφ9.2	0.090	d/4	56	4.00 (4.00)	7.45	4.59
	A19				Cφ17	0.208	100	182	6.75 (7.00)	13.35	12.80
	A20			800			d/4	165	7.00 (7.00)	13.85	12.80
	A21				Cφ19	0.260	100	191	7.50 (9.00)	14.50	15.50
	A22						d/4	207	10.00(10.00)	15.45	15.50
Series-B	B1		U				∞	110(-9)	4.00	7.70	6.10
	B2		B					113(-9)	4.40	7.80	6.10
	B3	L1	U		Bφ13	0.216	d/2	113(-3)	4.00	7.30	6.10
	B4		B					114(-9)	3.60	8.52	6.10
	B5		U				d/4	85(-19)	3.60	7.26	6.10
	B6		B	450				116(-7)	4.00	8.00	6.10
	B7		U		Bφ11	0.133		78	3.50 (3.20)	7.26	4.63
	B8		B					74	3.60 (3.60)	7.30	4.63
	B9	L2	U		Bφ13	0.216	d/4	102	3.60 (3.60)	7.60	6.10
	B10		B					103	3.20 (3.20)	8.20	6.10
	B11		U		Aφ17	0.241		121	5.10 (5.10)	9.08	7.88
	B12		B					138	5.20 (4.80)	10.33	7.88

*1) L1, L2: Without and with load reversals
*2) B: Bonded, U: Unbonded
*3) fc': Design compressive strength of concrete (kgf/cm^2)
*4) A, B, C and D indicate quality of prestressing bar:
A (nominal, fpy $\geq$ 80 kgf/mm^2), B (fpy $\geq$ 95 kgf/mm^2), C (fpy $\geq$ 110 kgf/mm^2), D (fpy $\geq$ 130 kgf/mm^2)
*5) s: Spacing of closed stirrup ties for lateral confinement
*6) σp: Prestress, ( ) indicate measured prestress at upper fiber
*7) ( ) indicate first flexural cracking loads in the negative direction

All of Series-A beams were uniformly prestressed over the section with two prestressing bars arranged symmetrically as shown in Fig. 1, and were grouted with cement paste of W/C = 45%.

A half number of Series-B beams, subjected to rversed loading, were uniformly prestressed as done in Series-A beams, whereas the remaining half number of beams were non-uniformly prestressed with one prestressing bar. And, one pair of beams, bonded and unbonded types, were prepared for each test variable.

Loading Tests

Totally 34 simply supported beams were tested under two patterns of cyclic loadings with or without load reversals. The former type of loading corresponds to L2 and the latter to L1 in Table 1.

In L1 type of loading the beams were fully unloaded at each repetition when the load attained the compression fiber strains of mid-span section (εc) of 0.001, 0.002, (0.0025), 0.003, the maximum ultimate (Pmax) and also some levels on the falling branch of load-mid-span deflection curve. On the other hand, in the beams of L2 type of loading each mid-span deflection amplitude were kept nearly equal in the positive and negative directions. Each amplitude corresponded to εc of 0.001, 0.002, (0.0025), 0.003 and Pmax, on the positive run, in the Series-A tests, and in addition in the Series-B tests to some levels on the falling branch curve.

TEST RESULTS AND DISCUSSIONS

Load Carrying Capacities

In all of the beams the first flexural cracking occurred when εc on the initial positive run was less than 0.001, and the negative flexural cracking strength was almost equal to the positive one as shown in Table 1. And, it was observed that the crack reopening loads did not significantly change with increase in the previous maximum attainable loads in all of the beams.

An unbonded beam (B7), having low steel index of q = 0.133 and narrow tie spacing of s = d/4, failed finally by rupture of prestressing steel at extremely large mid-span deflection of approximately 40 mm. The final failure in the other beams were caused by severe crushing of concrete without rupture of prestressing steel.

Table 1 shows that the measured maximum ultimate loads (Pmax) of beams under L2 type of cyclic loading are lower by at most 10% than those under L1 type of loading. Pmax of the unbonded beam was less by about 10% than that of the comparable bonded beam, irrespective of the loading types.

The measured ultimate loads of both bonded and unbonded beams, even after subjected to high intensity reversed loads, were greater than the calculations by assuming the rectangular stress block equal to fc' of concrete and actual yield strength (fpy) of prestressing steel, although the formers being somewhat less in the high strength concrete beams of fc'= 800 kgf/cm^2 with q = 0.260.

The significant shear cracks did not occur up to failure in all of the beams. This suggests that the requirement for web reinforcement as provided in the JSCE PC Code[2)] is conservative for the post-tensioned beam, whether it be bonded or unbonded, which is subjected to load reversals well into the inelastic range.

And then, it is indicated in Fig. 2 that the ratio of negative maximum load to positive one at each repeated cycle of attaining the same deflection amplitude (P-/Pt) tends to be smaller in the unbonded beams than the bonded ones, although the ratio being scarcely affected by the prestressing steel index.

Deformation Properties

1) Elastic Recovery

In Fig. 3 are shown some examples of elastic recovery on unloading, being expressed by restoring index (α) as defined in the term of ratio of restoring mid-span deflection to total one.

The indexes, α, were in excess of approximately 0.8 in all of the beams even when unloaded at Pmax.

It should, however, be noted that the elastic recovery commences to decrease rapidly in the high strength concrete beam of fc' = 800 kgf/cm^2 with wider tie spacing (s = 100 mm) once the cyclic load exceeds Pmax, although the tendency being not so evidently observed in the normal one having almost the same steel index as that. Another high strength beam (A21) having q = 0.260 showed the similar behavior. Therefore, the recommendation in New Zealand Code[1], that fc' shall not exceed 8000 psi (560 kgf/cm^2) unless special transverse reinforcement is provided, may be desirable for assuming enough elastic recovery in the prestressed beams under high intensity cyclic loads.

The unbonded beams showed, up to failure, somewhat marked elastic recovery in comparison with the bonded ones, as typically shown in Fig. 4.

2) Equivalent Coefficient of Damping

In Fig. 5 are shown the equivalent coefficients of damping (heq), which are determined from the hysteresis loops of load-mid-span deflection curves.

The coefficient, heq, commences to increase rapidly when the applied load reaches Pmax.

In the beams with narrower tie spacing of d/4, as shown in Fig. 5 (1), heq at the loading levels equal to or beyond Pmax decreases sharply with increasing the prestressing steel index when $q \leq 0.2$. At such high levels of cyclic loadings, however, the prestressed beam of fc' = 800 kgf/cm^2 and q = 0.260 has rather higher value of heq than that of fc' = 500 kgf/cm^2 and q = 0.20.

It should be also noted that heq at the falling branch range is relatively larger in the beam having wider tie spacing of s = 100 mm than the comparable beam with s = d/4.

The coefficients, heq, of the unbonded beams, as typically indicated in Fig. 5 (3), are considerably smaller at $\varepsilon c \leq 0.0025$ than those of bonded ones, but the effect of grouting on heq is not so evident at the loading levels beyond Pmax. This is due to the debonding of prestressing steel aggravated by high intensity load reversals[4], and confirms the previous results of tests that the energy absorption for grouted bars was only 10 to 20% greater than for ungrouted bars[5].

As far as Series-A, B tests are concerned, the value of heq at the maximum ultimate load, irrespective of spacing of tie and existence of grouting, is approximately 0.15 for q = 0.2 which may be considered appropriate as the upper limit in the seismic design of prestressed elements.

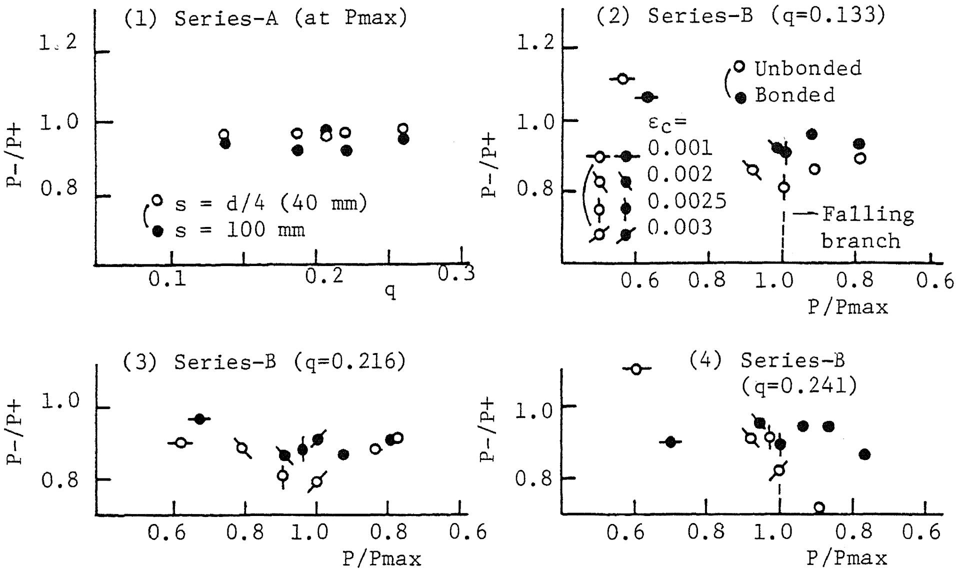

Fig. 2 Ratios of Negative Attainable Load (P-) to Positive One (P+) at Each Repeated Cycle

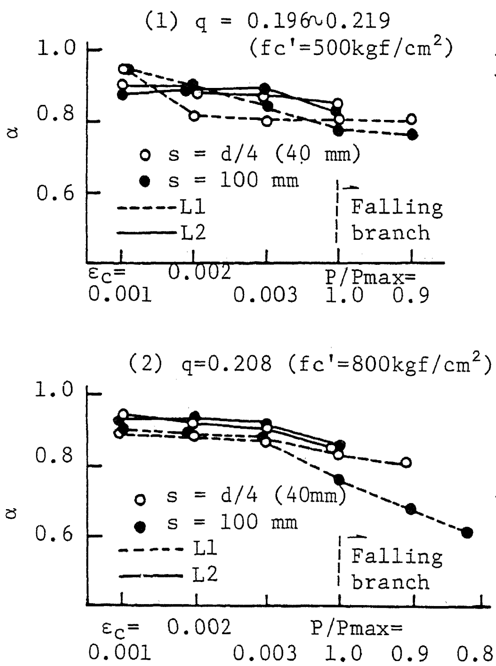

Fig. 3 Restoring Indexes (Series-A)

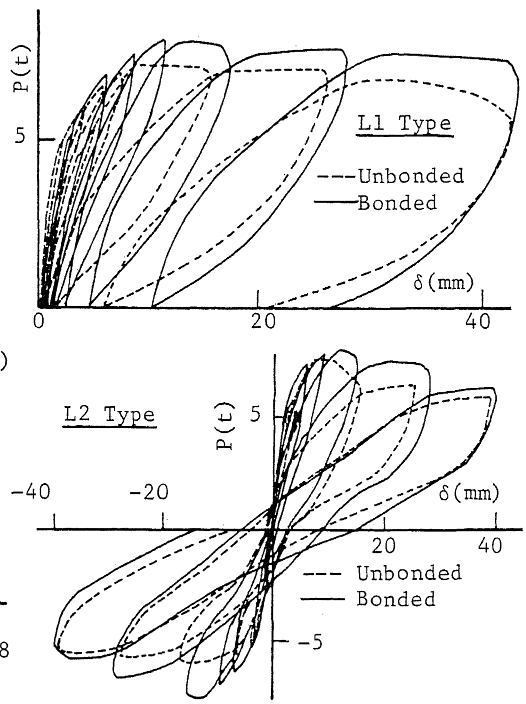

Fig. 4 Hysteresis Loops (Series-B: q=0.216)

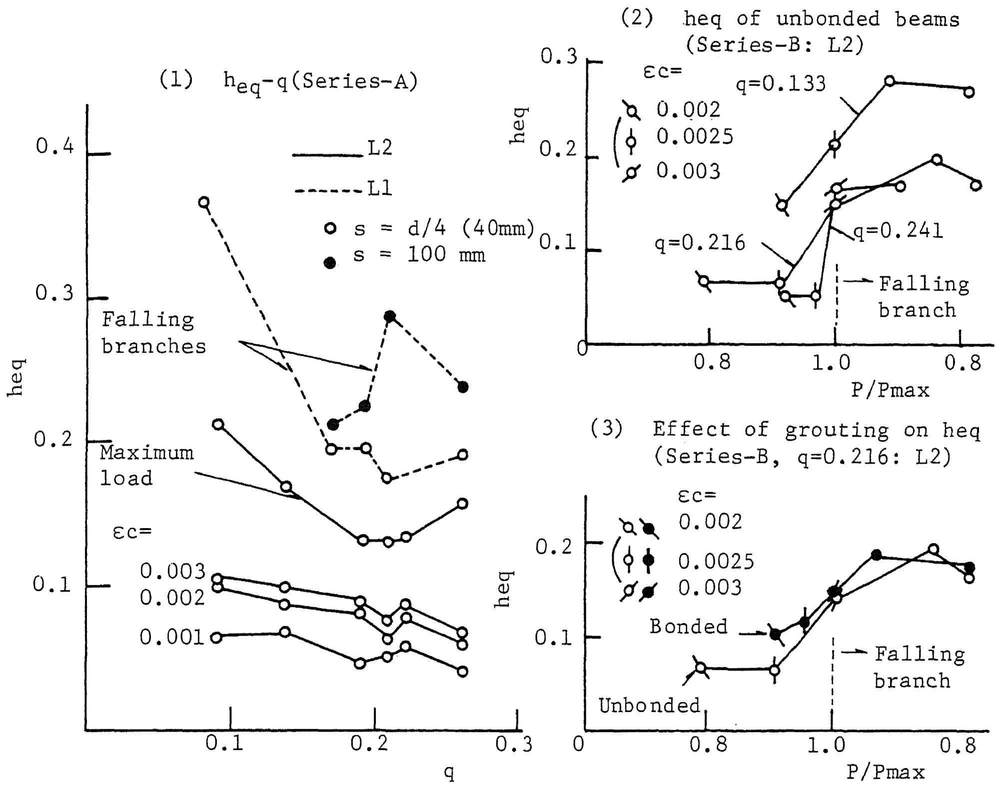

Fig. 5 Equivalent Coefficients of Damping

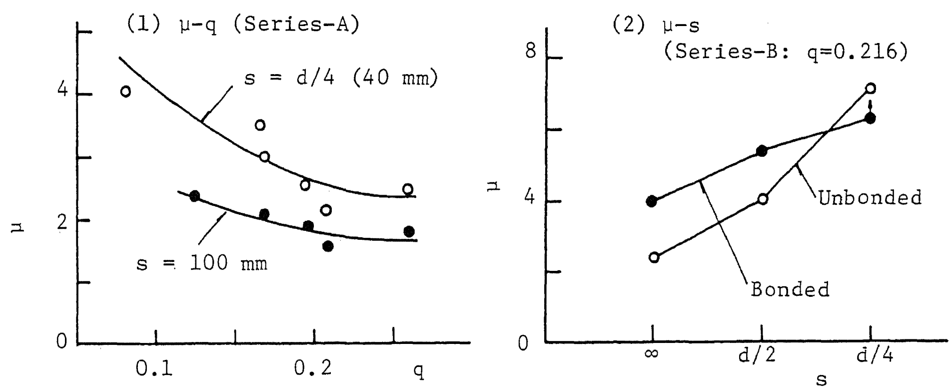

Fig. 6 Ductility Factors (L1)

3) Ductility Factors

The effects of prestressing steel index and grouting on ductility factor (μ) of beams are shown in Fig. 6 (1), (2), respectively. The ductility factor, μ, is defined in the term of ratio of two mid-span deflections at 0.9 Pmax on the ascending and falling branch curves given by an envelope of load-deflection hysteresis under L1 type loading.

It is clearly shown in Fig. 6 (1) that the ductility decreases considerably with increasing prestressing steel index in case of $q \leq 0.2$, and that it is also affected significantly by the spacing of tie.

As can be clearly seen in Fig. 6 (2), the unbonded beams show rather smaller ductility than the bonded ones when the tie spacing is relatively wide, for instance, $s \geq 1/2d$. And, therefore narrow arrangement of closed stirrup ties, $s = d/4$, is quite effective especially for improving the ductility of unbonded post-tensioned beams.

CONCLUSIONS

The main results of tests are summarized as follows.

1) Nearly 10% reduction in the maximum ultimate strength (Pmax) is expected to occur in the prestressed concrete beam after being subjected to high intensity reversed cyclic loadings. Pmax of unbonded beam, irrespective of loading patterns, is less by about 10% than that of bonded one.

2) The JSCE PC Code requirement for web reinforcement is adequate for preventing the shear failure of bonded and unbonded beams, even when the beams were subjected to reversed cyclic loads well into the inelastic range.

3) Elastic recovery of prestressed beams can be in excess of, at least, 80% even when the beams were unloaded at Pmax, and that is somewhat higher in the unbonded beams than the bonded ones. The beams made of fc' = 800 kgf/cm^2 and having wide tie spacing show significantly low elastic recovery at the loading levels beyond Pmax.

4) The equivalent coefficient of damping (heq) of beam at Pmax is equal to about 0.15 for $q = 0.2$ irrespective of tie spacing and existence of grouting, althouth affected considerably by the tie spacing within the falling branch range. And, the effect of grouting on heq does not become so evident after the applied load attains Pmax.

5) The closer tie spacing of, for instance, d/4 in the beams tested is very effective for improving the ductility of beams.

REFERENCES

1) Proposed Provisions for New Zealand Concrete Design Code: Chapter 22, Prestressed Concrete Members - Additional Seismic Requirements

2) Japan Society of Civil Engineers: Standard Code for Prestressed Concrete, 1978

3) Report of the FIP Commision on Seismic Structures, Proc. 7th Cong. of FIP, New York, 1974

4) Blakelay, R.W.G and Park, R.: Seismic Resistance of Prestressed Concrete Beam-Column Assemblies, Jour. of ACI, V.68, No.9, Sept. 1971

5) Kvitsaridze, O.I, et al: Experimental Study of Prestressed Reinforced Concrete Elements of Antiseismic Buildings, Proc. 6th WCEE, New Delhi, Jan., 1977