報　告

Comparative Study on Behavior of Pre-stressed & Reinforced Concrete Beams Subjected to Reversed Loading

By Shunji Inomata*

1. Introduction

Understanding of the behavior of pre-stressed concrete beams subjected to reversed loading and comparison of their behavior with that of the reinforced concrete beams are important because of the differing concept on pre-stressed concrete and reinforced concrete.

Behavior of a pre-stressed concrete member of class I (fully pre-stressed) or class II is more elastic than that of a reinforced concrete member under the effects of design loads and of other actions. Because of this complete elastic behavior of pre-stressed concrete beam, its energy-absorbing capacity is seemed to be inferior to that of the reinforced concrete beam and the pre-stressed concrete is generally supposed to be an unsuitable material for the seismic structure.

If a structural pre-stressed concrete member should keep up the behavior of class I or II even under an exceptional superimposed load of short duration such as abnormal earthquake, the member would be too much elastic to be able to absorb enough energy and also such member would have an excessive safety against the ultimate failure.
But in the case of reinforced concrete member, allowable stresses are generally allowed to be increased by 50% under such exceptional superimposed load and many large cracks my be expected in the member resulting in great plastic deformation. Because of this fact the reinforced concrete member can absorb much energy, if it is well designed, however the safety against ultimate failure is rather small under such abnormall load.

Considering these circumstances the energy-absorbing capacity could be increased without reducing the necessary safety against failure if pre-stressed concrete member is allowed to become to the state of class III under the severe seismic motion.

The present paper reports experimental results on 12 beams with the parameters studied given in **Table 1**.

To make a comparison between the behavior of pre-stressed concrete beam and that of reinforced concrete beam under the reversed loading, two cases were studied; (a) beams with the same ultimate failure strength, (b) beams with the same working strength. The working load was calculated as the decompression load for pre-stressed concrete beam and was estimated using the conventional allowable stresses for reinforced concrete beam.

2. Test Specimens

All beams are of rectangular section and has a stub at midspan giving a simulated beam-column connection. **Fig. 1** shows principal dimensions of the test beams and their marks are given in **Table 1**.

Table 2 shows the cross-section of the test beams at the connection between beam and stub and also shows the properties of the materials used in the test beams. Stress-strain relation of the

Table 1 Mark and Type of Test Beams

Beam No.	Type	Remarks
R-I, R-II P-I, P-II	Reinforced Concrete Prestressed Concrete	Same working load
R-A $P\text{-}A_b$	Reinforced Concrete Prestressed Concrete	Same failure load
R-B-1, R-B-2 $P\text{-}B_b\text{-}1$, $P\text{-}B_b\text{-}2$ $P\text{-}B_{ub}\text{-}1$, $P\text{-}B_{ub}\text{-}2$	Reinforced Concrete Prestressed Concrete Unbonded Prestressed Concrete	Same failure load

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Tabie 2 Cross Section at Colume Face & Mechanical Properties of Materials

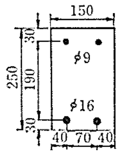
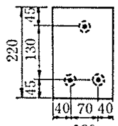
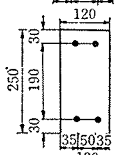
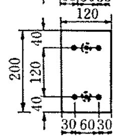
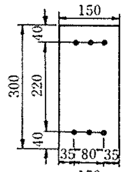
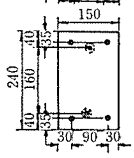
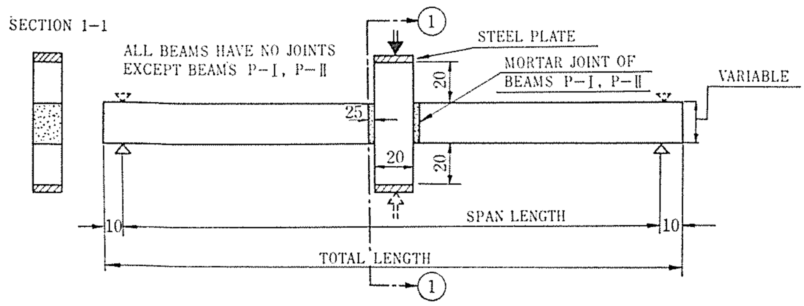
<table>
<tr><th>Beam No.</th><th colspan="2">Cross Section</th><th>Mechanical Properties of Materials</th></tr>
<tr><td>R-Ⅰ
R-Ⅱ</td><td></td><td>Reinforced Concrete
two round bars ϕ 9
two round bars ϕ 16</td><td>Concrete $\sigma_{cu}=383$ kg/cm²
Round bar<table><tr><th>Diameter (mm)</th><th>σ_{sy} (kg/mm²)</th><th>σ_{su} (kg/mm²)</th></tr><tr><td>9</td><td>33.0</td><td>46.6</td></tr><tr><td>16</td><td>33.9</td><td>45.2</td></tr></table></td></tr>
<tr><td>P-Ⅰ
P-Ⅱ</td><td></td><td>Prestressed Concrete
three tendons ϕ 11
no round bar (grouted)</td><td>Concrete $\sigma_{cu}=525$ kg/cm²
Tendon
Diameter…………11.2 mm
σ_p 0.2 …………126 kg/mm²　σ_{pu}……………138 kg/mm²</td></tr>
<tr><td>R-A</td><td></td><td>Reinforced Concrete
four round bars ϕ 16</td><td>Concrete $\sigma_{cu}=260$ kg/mm²
Round bar
σ_{sy} ……………31.4 kg/mm²
σ_{su} ……………43.0 kg/mm²</td></tr>
<tr><td>P-A$_b$</td><td></td><td>Prestressed Concrete
two tendons ϕ 10
four round bars ϕ 9
(grouted)</td><td>Concrete $\sigma_{cu}=400$ kg/cm²
Round bar
σ_{sy} ……………31.7 kg/mm²
σ_{su} ……………43.2 kg/mm²
Tendon
Diameter ………10.1 mm
σ_p 0.2…………130 kg/mm²
σ_{pu} ……………147 kg/mm²</td></tr>
<tr><td>R-B·1
R-B·2</td><td></td><td>Reinforced Concrete
six round bars ϕ 13</td><td>Concrete $\sigma_{cu}=357$ kg/mm²
Round bar
σ_{sy} ……………31.2 kg/mm²
σ_{su} ……………43.2 kg/mm²</td></tr>
<tr><td>P-B$_b$·1
P-B$_b$·2</td><td rowspan="2"></td><td>Prestressed Concrete
two tendons ϕ 10
four round bars ϕ 9 (grouted)</td><td>Concrete $\sigma_{cu}=464$ kg/cm²
Round bar
σ_{sy} ……………30.2 kg/mm²
σ_{su} ……………43.4 kg/mm²
Tendon
Diameter ………10.1 mm</td></tr>
<tr><td>P-B$_{ub}$ 1
P-B$_{ub}$ 2</td><td>two tendons ϕ 10
four round bars ϕ 13
(non-grouted)</td><td>Concrete $\sigma_{cu}=478$ kg/cm²
Round bar
σ_p 0.2 ………125 kg/mm²　σ_{sy} …………31.2 kg/mm²
σ_{pu} ……………142 kg/mm²　σ_{su} …………43.2 kg/mm²</td></tr>
</table>

Fig. 1 Principal Dimensions



Beam No.	Total Length	Span Length
R-Ⅰ, R-Ⅱ, P-Ⅰ, P-Ⅱ	280 cm	260 cm
R-A, P-A$_b$, R-B, P-B$_b$, P-B$_{ub}$	260 cm	240 cm

prestressing steels is given in **Fig. 2**.

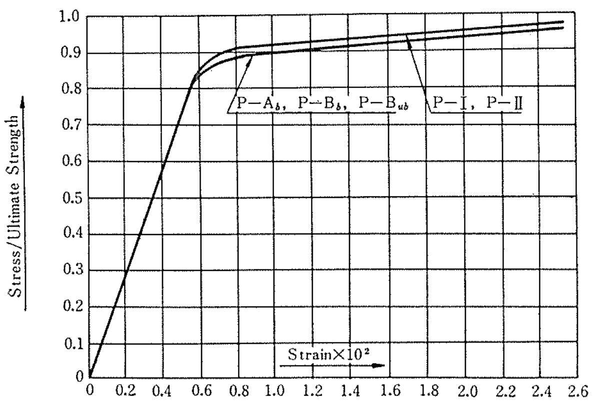
All beams except P-Ⅰ and P-Ⅱ were cast monolithicly, whereas P-Ⅰ and P-Ⅱ beams were cast in three pieces; two parts of the beam and one stub piece were assembled by longitudinal prestressing after hardening of mortar joints of 25 mm thick between beams and stub. All pre-stressing bars were anchored against the concrete beams using threads and nuts and bonded to the concrete injecting cement paste into the sheath after the transfer of pre-stress, expect P-B$_{ub}$ beams. Initial pre-stressing force for each pre-stressing steel bar was 8.5 t for P-Ⅰ and P-Ⅱ beams, 7.5 t for P-A$_b$ beam and 7.3 t for P-B$_b$ and P-B$_{ub}$ beams, respectively.

The working bending moment and ultimate failure moment of each beam at the stub face were calculated on the following assumptions.

(1) **Working bending moment**

For the reinforced concrete beam, the working bending moment was determined appling the conventional method in which modulus raition and the permissible stresses for concrete and steel were assumed to be equal 15,80 kg/cm² and 1 600 kg/cm²,

Fig. 2 Stress-Strain Curve



respectively.

For the pre-stressed concrete beam, the bending moment at a decompression limite state on a tensile fiber was taken as the working moment.

(2) **Ultimate failure bending moment**

The analysis for ultimate failure moment was baised on the following assumptions, (a) maximum unit strain of the concrete due to compression in simple bending was taken as 0.0035, (b) the diagram representing the stress distribution in the compressed zone was taken to be a rectangule and the width of the rectangle was equal to the measured mean strength on cylinder (σ_{cu}), its height was taken to be 0.75 tims the depth X of the zone subjected to compression, (c) the determination of the strain in the steel was done on the basis of its measured stress-strain relationship, (d) the possible increase of tensile stress of the unbonded prestressingsteel above the effective tensile stress was assumed to be equal 1 400 kg/cm².

3. Test Results

All test beams were loaded at the midspan through the stub as shown in the drawing of the specimen in **Fig. 1**. All beams were subjected to several reversal of loading, i. e. at first load was gradually increased up to a certain fixed load level and then reduced to zero load, repeating three times these loading and unloading cycles up to the same load level in one direction, and then the loading direction was reversed repeating the same number of loading and unloading cycles up to the load level which was attained in the previous loading direction. The load level was increased after finishing above-mentioned one complete loading cycle and the same loading procedures were followed. After several cycles of this loading process, the load was incressed until the beam failed in the original loading direction.

In order to observe the development of cracks and to measure the deflection and rotation of section at the mid-span of the beam, the loading was done slowly, taking about 30 minutes to perform one complete reversed loading cycle.

Deflection at the mid-span of the beam was measured by dialgauge placed under the stub and at the same time the rotation of the beam section against the stub face was measured using the devices shown in **Fig. 3**.

Using the previously calculated working and ultimate failure bending moment, load P which conld be applied to the beam through the stub was obtained for each test beam excluding the effect of the dead weight. These results were shown in **Table 3** and at the same time this Table gave the cracking load for the pre-stressed concrete test beam and also the load calculated on the assumption of the increased permissible stresses by 50%

Fig. 3 Device for Measuring Rotation

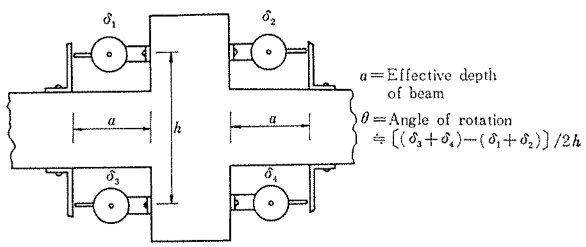


Table 3 Calculated Loads (kg)

Beam No.	P_w (1)	$P_{1.5w}$ (2)	P_{cr} (3)	P_u (4)
R-I	+ 1 850	+ 2 860		+ 4 530
R-II	− 490	− 820		
P-I	+ 2 080		+ 3 100	+ 7 000
P-II	− 410		− 1 450	
R-A	± 2 080	± 3 030		± 4 260
P-A_b	± 810		± 1 640	± 4 300
R-B	± 2 430	± 3 740		± 6 000
P-B_b	± 910		± 2 590	± 5 780
P-B_{ub}	± 910		± 2 590	± 6 030

(1) Working load
(2) Load when permissible stresses are increased by 50%
(3) Cracking load
(4) Ultimate failure load

Table 4 Cracking and Failure Loads
(Mean Value)

(kg)

Beam No.	Cracking load			Failure load		
	Measured	Calculated	Measured / Calculated	Measured	Calculated	Measured / Calculated
R-I, R-II	+ 1 120 − 490			+ 4 450	+ 4 530	0.98
P-I, P-II	+ 3 400 − 1 360	+ 3 180 − 1 450	1.07 0.94	+ 7 310	+ 7 000	1.04
R-A $P-A_b$	+ 1 200 + 2 000	 + 1 640	 1.22	+ 4 800 + 4 300	+ 4 260 + 4 300	1.12 1.00
R-B $P-B_b$ $P-B_{ub}$	+ 800 + 2 400 + 2 500	 + 2 590 + 2 590	 0.93 0.96	+ 6 100 + 6 050 + 6 350	+ 6 000 + 5 780 + 6 030	1.02 1.05 1.05
		Mean…………	1.02		Mean…………	1.04

for concrete and steel according to the general practice now used for the reinforced concrete structure while subjected to seismic effects.

Measured cracking and ultimate failure loads are given in **Table 4**, showing that cracking and ultimate failure loads have scarcely been affected by the repeated reverse loading.

Typical test results on load-deflection and load-rotation relationships are given in the **Fig. 4～14.**

Ovserved crack pattens are shown in **Fig. 15** and **16,** inscribing the value of load at which each crack was observed during loading. At the connection between beam and stub, they were separated by cracks through the whole section near the final loading cycle.

With reference to the spacing of cracks, the beam P-I, P-II having mortar joint between beams and stub showed greater spacing of the order of 16～19 cm than that of the beams R-I, R-II in which the spacing of cracks was about 14 cm. The beams R-A and $P-A_b$ had almost same mean spaing of 10 cm but much more cracks were observed in the beam R-A than in the beam $P-A_b$, showing 16 craeks against 11 cracks. The mean spacing of cracks was 15 cm, 11 cm and 13 cm in the beams R-B, $P-B_b$ and $P-B_{ub}$, respectively.

Fig. 4 Load-Deflection Curve of Beam R-II

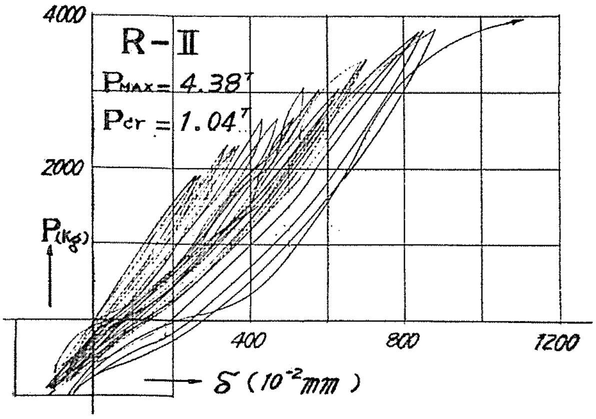


Fig. 5 Load-Deflection Curve of Beam P-I

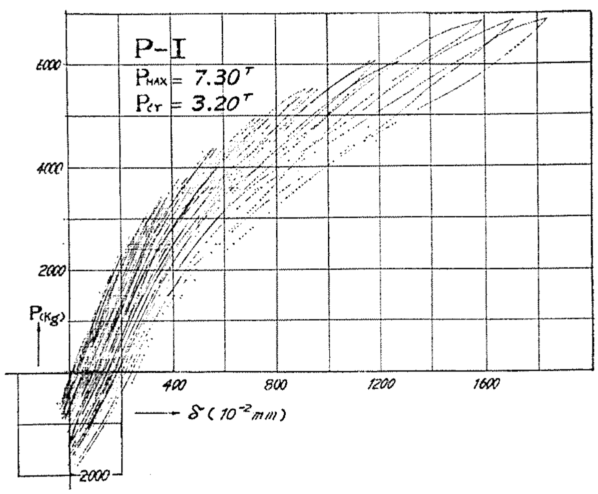


Fig. 6 Load-Rotation Curve of Beam P-I

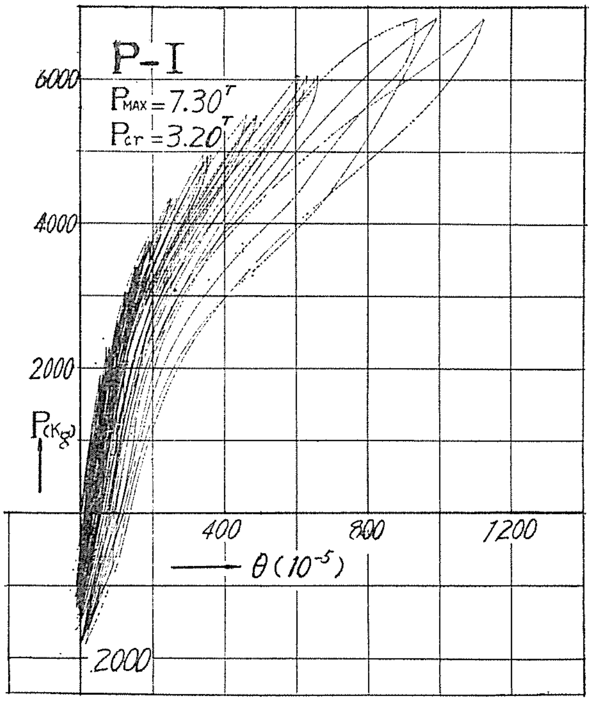


Fig. 7 Load-Deflection Curve of Beam R-A

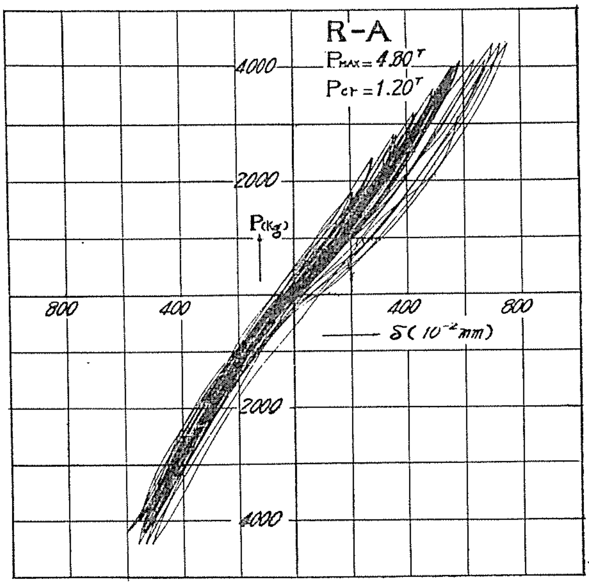


Fig. 8 Load-Rotation Curve of Beam R-A

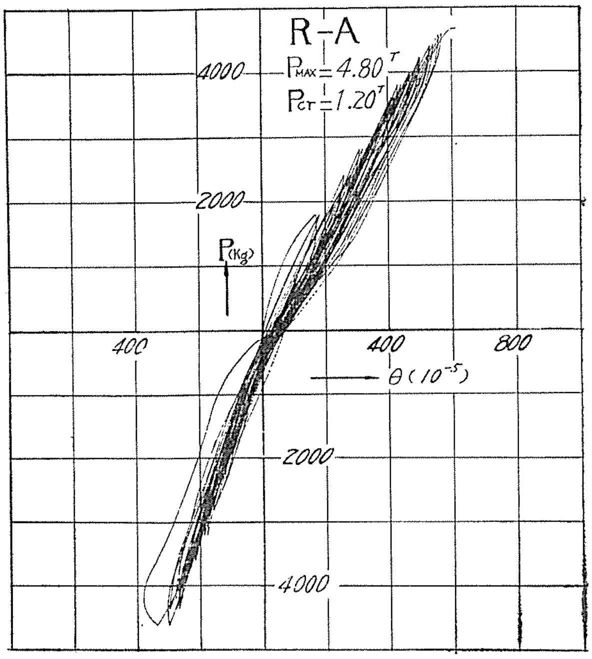


Fig. 9 Load-Deflection Curve of Beam P-A_b

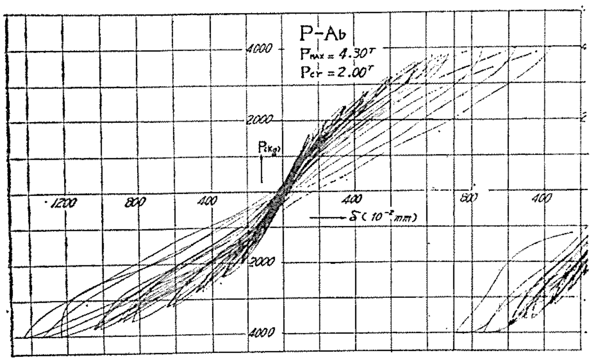


Fig. 10 Load-Rotation Curve of Beam P-A_b

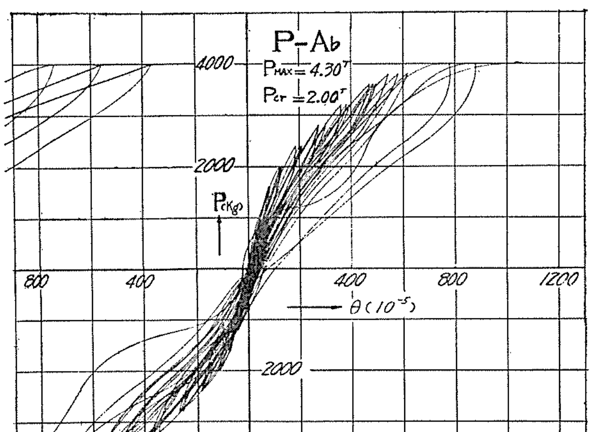


Fig. 11　Load-Deflection Curve of Beam R-B·2

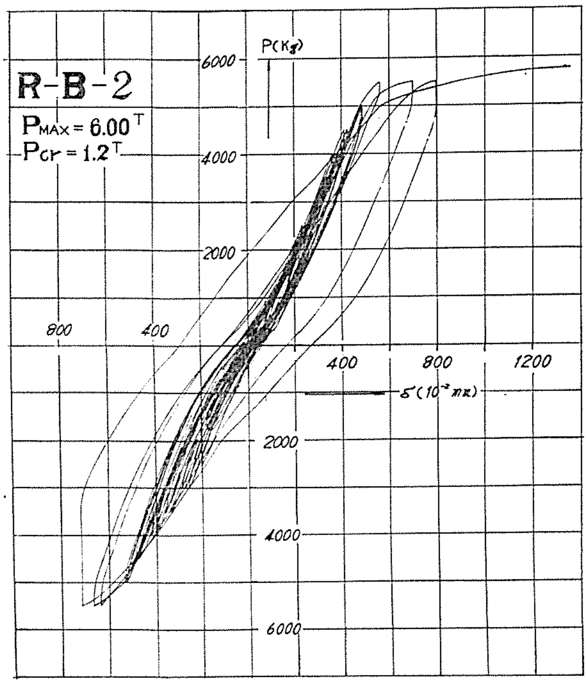


Fig. 13　Load-Deflection Curve of Beam P-B_{ub}-1

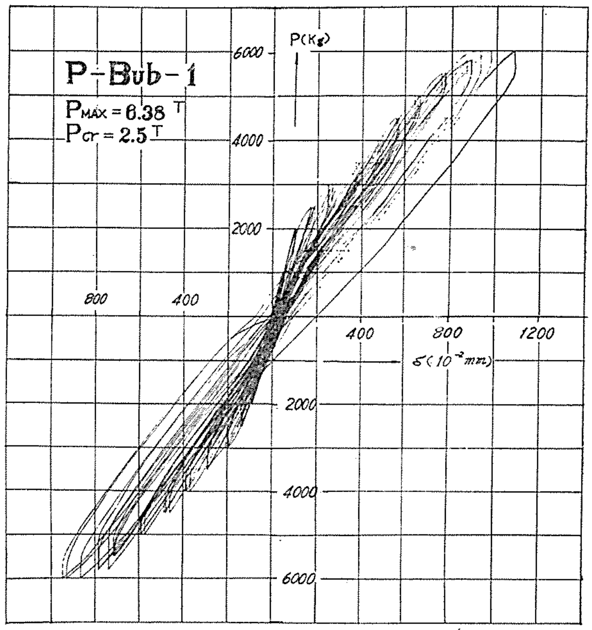


Fig. 12　Load-Deflection Curve of Beam P-B_b·1

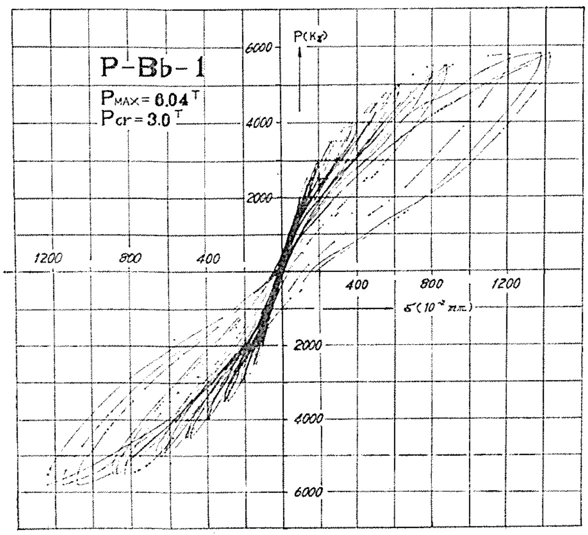


Fig. 14　Load-Deflection Curve of Beam P-B_{ub}-2

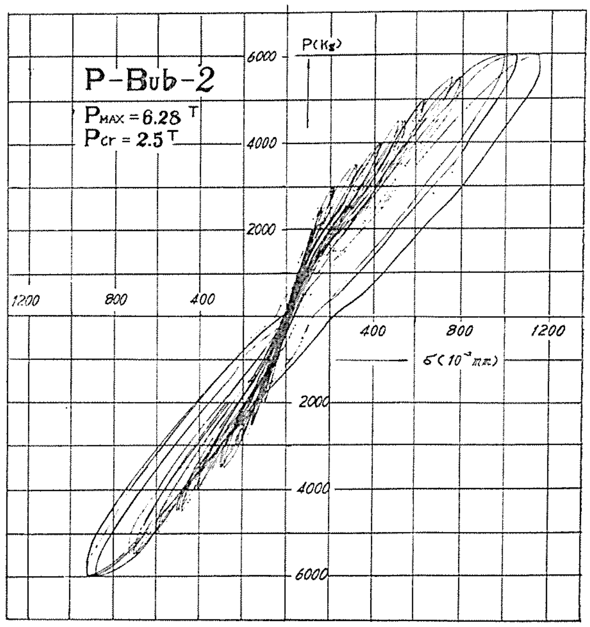


Fig. 15　Crack Pattern

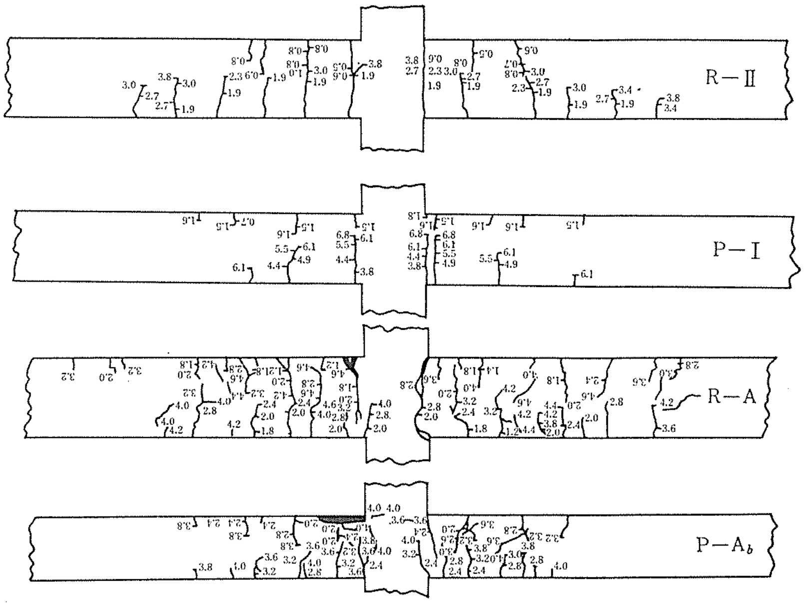


Fig. 16　Crack Pattern

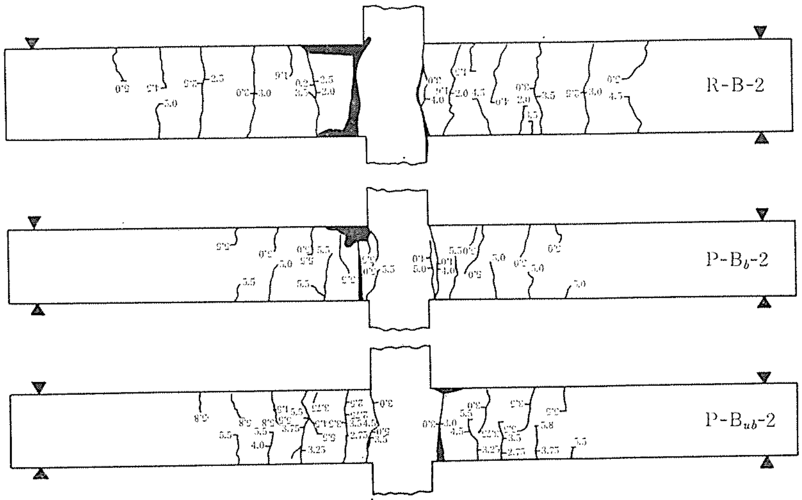
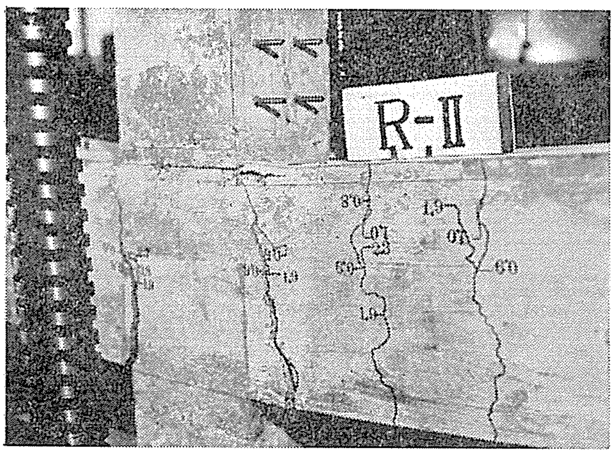


Fig. 17　Close-up of Crashed Section of Beams

（1） R-Ⅱ



（2） P-Ⅱ

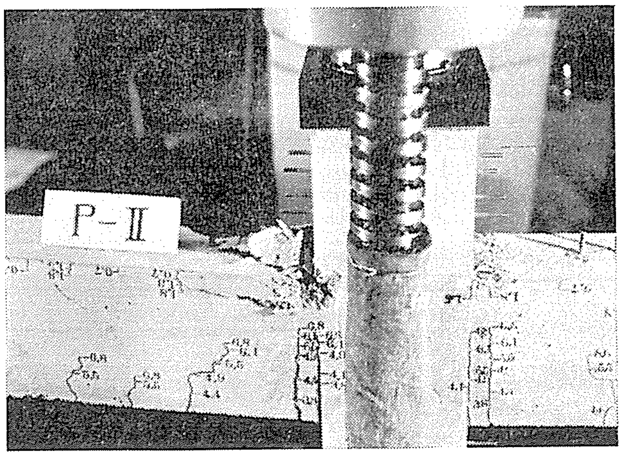


Fig. 17 Close-up Crashed Section of Beams
(3) P-A_b

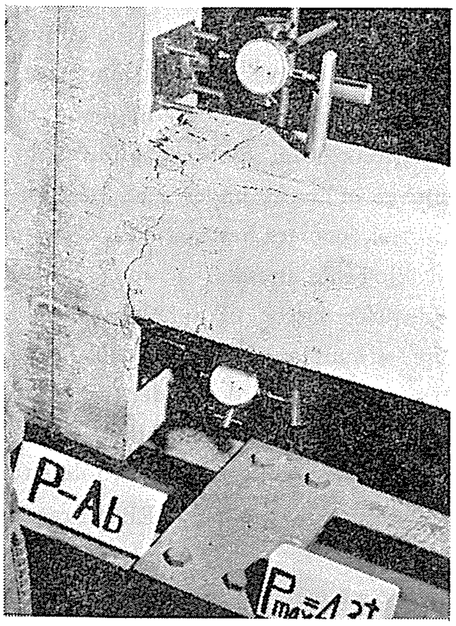


Fig. 18 Close-up of Crashed Section of Beams
(1) R-B·1

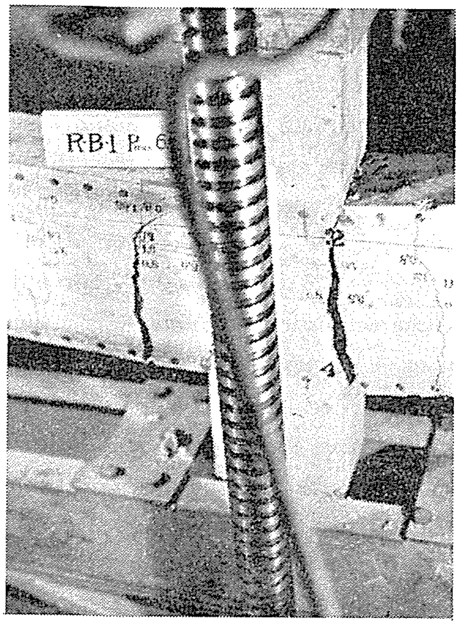
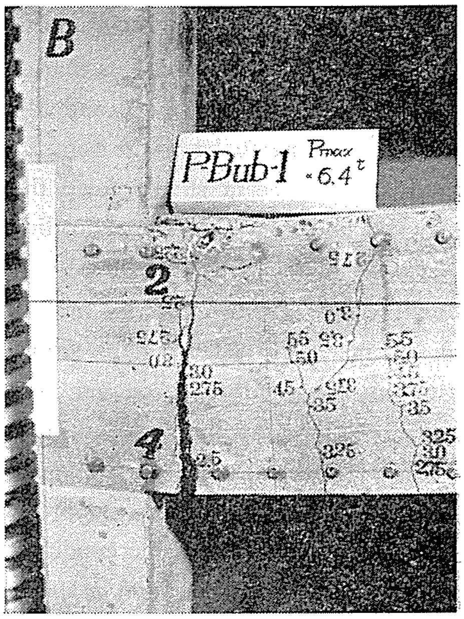
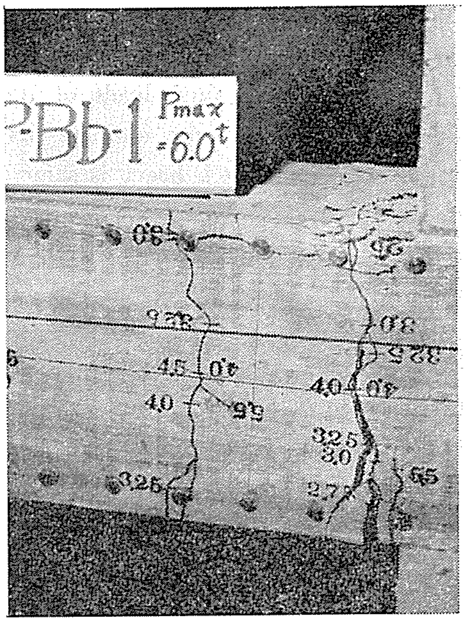


Fig. 18 Close-up of Crashed Section of Beams
(2) P-B_b·1



(3) P-B_{ub}·1



All beams failed as a result of the crushing of concrete on the compression face at the sections between beams and stub where the through cracks were observed before the final loading.

Some typical failure conditions were shown in **Fig. 17** and **18**.

4. Discussion

It is difficult to draw out any complete conclusion on the comparative behavior of the pre-stressed concrete beam and the reinforced concrete beam subjected to reversed loading for the range of beams tested in this report. Therefore only general aspects obtained from the test results will be discussed hereinafter.

(1) For a given working load, a pre-stressed concrete designed for class I has greater factor of safety against failure than a reinforced concrete designed by the conventional method.

The reinforced concrete beams R-I and R-II give mean factor of safety against failure of 2.4, whereas the pre-stressed concrete beams P-I and

P-II give that of 3.5. The allowable stresses for concrete and steel being increased by 50% as usually permitted in the case of earthquakes, the factor of safety against failure is less than 1.6 for the reinforced concrete beams R-I and R-II. Therefore there seems to be no reasons that the pre-stressed concrete beam should keep the state of class I even under an exceptional loading such as severe earthquakes.

If the allowable stresses could be increased by 50% and less margin of safety against failure may be allouwed in the reinforced concrete structures under such exceptional loading, only ultimate limit state should be checked to keep an appropriate factor of safety against failure in the pre-stressed concrete structures under the same condition. But a limit state of elongation of tendon or steel should be considered to prevent excessive cracks and loss of pre-stress after ceasing of earthquake motions that might occur several times during the life of the pre-stressed concrete structures.

(2) Comparing the failure loads of the test beams having the almost same ultimate strength, the factor of safety for each beam is summerized in **Table 5**.

It can be seen that the ultimate failure load is about 2.4 and 1.6 times as large as the working load and the calculated load assuming the possible increase of the allowable stresses, respectively in the reinforced concrete beams. The ultimate failure load of the pre-stressed concrete beams is at least 5.3 and 2.3 times as large as the working load of class I pre-stressed concrete and the initial cracking load.

Table 5 Factor of Safety for Failure

Beam No.	Ultimate P_u (kg)	Working P_w (kg)	P_u/P_w	$P_{1.5w}$(2) or (kg) P_{cr} (kg)	$P_u/P_{1.5w}$ or P_u/P_{cr}
R-A	4 800	1 980	2.42	3 030	1.59
R-B	6 100	2 430	2.51	3 740	1.63
P-A_b	4 300	810	5.31	1 640	2.62
P-B_b	6 050	910	6.65	2 590	2.33
P-B_{ub}	6 350	910	6.97	2 590	2.45

(1) Working load, calculated using the allowable stresses 80 kg/cm² for concrete and 1,600 kg/cm² for steel in the beams R-A and R-B; calculated at the state of decompression in the beams P-A_b, P-B_b and P-B_{ub}.

(2) The load, calculated on the assumption that the allowable stresses may be increased by 50% in the beams R-A and R-B; calculated at the state of initial cracking in the beams P-A_b, P-B_b and P-B_{ub}.

These figures obtained from the tested pre-stressed concrete beams may be too conservative in the case of exceptional loading and the pre-stressed concrete beam could be allowed to develop a number of cracks under such load for which the reinforced concrete beam is designed using the increased allowable stresses.

The adequate factor of safety against failure being kept, a number of cracks should be accepted in the pre-stressed concrete under an exceptional load such as severe earth-quake, unless these cracks do not impair the stability of the whole structure.

(3) According to the FIP-CEB "Practical recommendation for the design and construction of pre-stressed concrete structures" the flexural ultimate limit strength should be calculated considering the design strength of the materials and limitting possible steel elongation less than 1%. In calculating the ultimate limit strength of the test beams, the reduction factor γ_m was assumed to be equal to 1.4 and 1.15 for concrete and steel, respectively, and the characteristic strength adopted in the calculation was chosen from the compressive strength of concrete and tensile strength of prestrenssing steel shown in **Table 2**. For ordinarly steel the characteristic yield point was assumed to be equal to 30 kg/mm².

The parabolic-rectangular diagram was used to determine the stress distribution for the compressed zone of the concrete. In this diagram the top of the parabola occurs at the abscissa 0.2% strain and the extreme angle of the rectangle at 0.35% strain.

In **Table 6** the calculated ultimate limit load P_u^* and the measured failure load P_u are shown to compare these two values.

Table 6 Calculated Ultimate Limite Load P_u^*

Beam No.	P_u^* (kg)	P_u (kg)	P_u/P_u^*
R-I, R-II	3 380	4 450	1.32
R-A	3 600	4 800	1.33
R-B	4 400	6 100	1.49
P-I, P-II	5 080	7 310	1.44
P-A_b	2 760	4 300	1.55
P-B_b	4 000	6 050	1.51

The measured failure load was at least 1.3 and 1.4 times as large as the calculated ultimate limit load for the reinforced concrete beams and the pre-stressed concrete beams, respectively.

Ultimate limit load being determined according to the design method of the FIP-CEB recommendation, the test beams can gurantee sufficient margin of safety against an exhaustion of load carring capacity.

The above-mentioned ultimate limit load being able to be taken as a criterion for safety against failure of beam, the ratio between the measured deflection or rotation at the critical section under the failure load P_u and that under the calculated ultimate limit load $P_u{}^*$ could be defined as a ductility factor which can show the possible deformation after the working load reaches the load $P_u{}^*$.

The final deflection or rotation at the critical section could not be measured with great accuracy because of keeping the instruments from the expected damages.

The ratio between the measured deflection under the maximum measured load P_u and that of the ultimate limite load P_u is given in **Table 7.**

The possible final deformation was from 2.5 to 2.8 and from 3.8 to 5.7 times as large as the deformation corresponding to the ultimate limit load $P_u{}^*$ for the reinforced concrete beams and the pre-stressed concrete beams, respectively.

The energy-absorbing capacity of the test beams

Table 7 Ductility Factor μ_δ

Beam No.	Measured deflection (mm)		$\mu_\delta=\delta_u/\delta_u{}^*$
	δ_u	$\delta_u{}^*$	
R-I, R-II	16.24	6.62	2.5
R-A	15.37	5.43	2.8
R-B	12.30	4.62	2.8
P-I, P-II	23.44	6.25	3.8
P-A_b	22.58	4.60	4.9
P-B_b	23.11	4.08	5.7
P-B_{ub}	23.45	4.38	5.4

δ_u=deflection at failure
$\delta_u{}^*$=deflection at calculated ultimate limit. Because of the unbonded tendons, the ultimate limit load $P_u{}^*$ of the beam P-B_{ub} can not be calculated by the same method adopted in the other prestressed concrete beams, but for simplicity the calculated limit load 4 000 kg is taken for the beam P-B_{ub} as for the beam P-B_b.

Fig. 19 Determination of Energy-Absorbing Rate

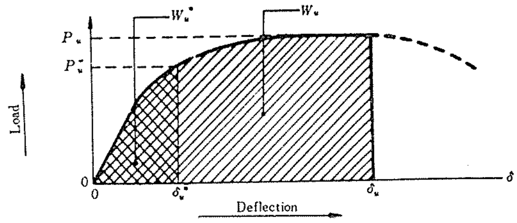


Table 8 Ratio $W_u/W_u{}^*$

Beam No.	Mean area		$W_u/W_u{}^*$
	$W_u{}^*$	W_u	
R-I, R-II	260	1 065	4.1
R-A	206	1 117	5.4
R-B	215	1 122	5.5
P-I, P-II	391	2 669	6.9
P-A_b	162	1 551	9.6
P-B_b	212	2 340	11.1
P-B_{ub}	232	2 467	10.7

may be represented by the area on the load and deflection curve.

The ratio between the area (W_u) on the P-δ curve at failure and that ($W_u{}^*$) at the limit load $P_u{}^*$ may be considered as a criterion for the possible energy-absorbing capacity of the beam. These areas W_u and $W_u{}^*$ are represented in **Fig. 19**.

The measured ratio between W_u and $W_u{}^*$ is given in **Table 8**. The test results show that the ratio $W_u/W_u{}^*$ is equal to 4.1–5.4 and 6.9–11.1 for the reinforced concrete beams and the pre-stressed concrete beams, respectively.

Considering these test results shown in **Table 7** and **Table 8**, it can be said that the pre-stressed concrete beams have always greater final deformation and energy-absorbing capacity than the reinforced concrete beams, when the ultimate limit load calculated by the FIP-CEB recommendation is taken as a criterion for safety against the failure of the test beams. Also it is clear that the final deformation and energy-absorbing capacity of the pre-stressed concrete beam could be increased by putting more longitudinal ordinary steel bars into the cross-section.

(4) Feed-in energy from loading less energy feed-back from unloading must equal energy lost to heat plus dissipation in work or damage done.

This dispersed energy may be correlated with damping characteristic of the test beam and could be represented by the area on the P–δ curve deducting the area of un-loading from that of loading.

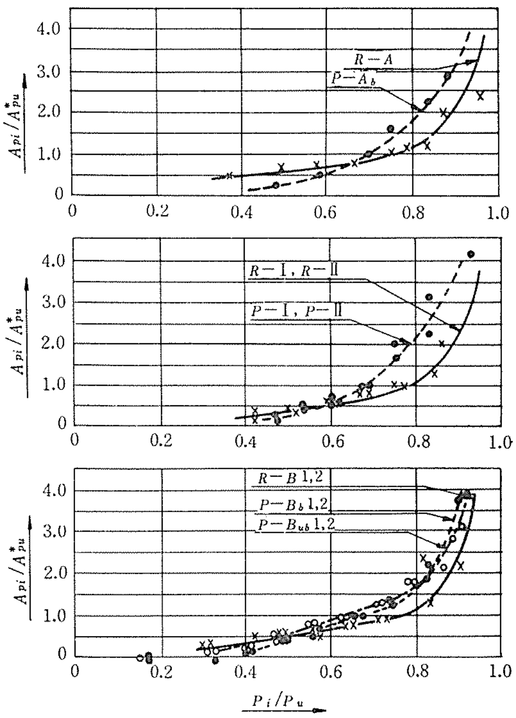
The above-mentioned area $A_{pu}{}^*$ at the calculated ultimate limit load beidg taken unity, the ratio between the area A_{pi} at any load P_i and the area $A_{pu}{}^*$ is shown in **Fig. 20**. From these figures it can be seen that the pre-stressed concrete beams have less energy dissipation so far as the applied load remains smaller than the ultimate limit load $P_u{}^*$, but over this ultimate limit load they have always greater energy dissipation than the reinforced concrete beams.

The ratio of the above-mentioned area $A_{pu}{}^*$ on the P–δ curve between reinforced concrete and pre-stressed concrete beams in each test group is given in **Table 9**.

Table 9 Ratio of Area $A_{pu}{}^*$

Beam No.	Ratio of Area $A_{pu}{}^*$
R-I, R-II	1.00
P-I, P-II	0.64
R-A	1.00
P-A_b	0.64
R-B	1.00
R-B_b	0.72
R-B_{ub}	0.78

Fig. 20 Comparison of Dispersed Energy



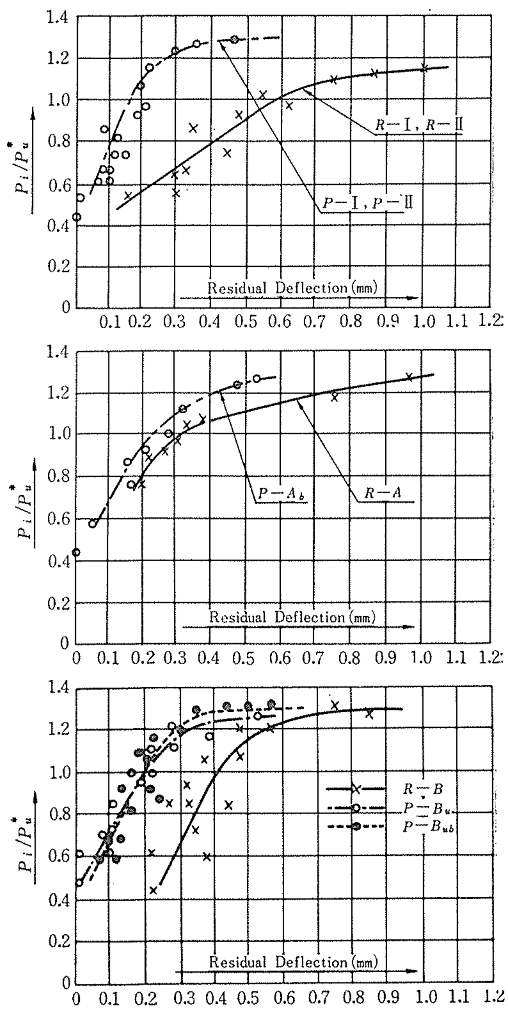
The reinforced concrete beams have always greater energy dissipation at the ultimate limit load $P_u{}^*$, but the rate of energy dissipation could be increased for the pre-stressed concrete beams by putting the adequate amount of longitudinal ordinary steel into the cross-section.

(5) The relationship between the measured residual deflection after a certain load P_i being reduced to zero at each loading stage and the ratio $P_i/P_u{}^*$ is shown in **Fig. 21**.

At the same ratio $P_i/P_u{}^*$, the reinforced concrete beams give greater residual deflection than the pre-stressed concrete. A linear relationship between the residual deflection and the ratio $P_i/P_u{}^*$ could be assumed to keep up till the ratio $P_i/P_u{}^*$ comes to unity.

All test beams have a same tendency of giving

Fig. 21 Relationship between Residual Deflection and Ratio $P_i/P_u{}^*$



a sudden increase of residual deflection, when the ratio P_i/P_u^* becomes greater than 1.0 or 1.1.

This possible sudden increase of residual deflection may be attributed to one of the causes which give rise to an alternative plasticity or incremental collapse of the reinforced or pre-stressed concrete structures under the effects of variable repeated loading such as severe earthquake motions. Therefore it seems to be advisable that the reinforced or pre-stressed concrete structures should be designed so as to keep the effects of the variable repeated loading of short duration from reaching the ultimate limit load carring capacity P_u^*.

5. Conclusions

For the range of beams tested, it appears that unless the first damaging load reaches about 80 % of the collapse load, the capacity in the reverse direction will be only slightly impaired, if at all, and a prestressed concrete structure or a part of it needs not be designed for class II or III under an exceptional loading of short duration but should be designed so as to fulfil the safety conditions as regards rupture Under such loading the behavior of a pre-stressed concrete beam can be improved only by placing adequate amount of longitudinal reinforcement.

To provide adequate ductility and reduce excessive loss of rigidity, the ultimate limit load of a pre-stressed concrete member should be calculated in accordance with the FIP-CEB recommendation. The pre-stressed concrete member designed with this method seems to have sufficient ductility comparable with that of the reinforced concrete member.

Finally, the design consideration which keeps the effects of external load away from the calculated ultimate limit load carring capacity is of great help in avoiding excessive residual deformation in a structure or a part of it.

6. Acknowledgment

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