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Earthquake response characteristics of prestressed concrete building structures

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ABSTRACT: A hysteretic model for prestressed concrete structures was proposed based on the past experimental studies. The increased response of prestressed concrete structures compared with reinforced concrete structures were demonstrated by nonlinear dynamic response analyses on single and multi degree-of-freedom system with the proposed model. And nonlinear dynamic response analyses using proposed hysteretic model traced the response of the model frame specimens subjected to the pseudo dynamic test loading with reasonable accuracy.

1 INTRODUCTION

The effect of hysteretic energy dissipation characteristics of prestressed concrete (hereinafter referred to as PC) members on the earthquake response characteristics of PC building structures has been discussed since 1960's. At the third WCEE held in New Zealand in 1965 some discussions were made on the energy dissipation characteristics of PC members. Mr. Despeyroux pointed out two interesting comment(Ref.1); 1)Though the energy dissipation of PC members in elastic range was lower than that of reinforced concrete members, in ultimate stage, the energy dissipation of both members became almost same. 2)The ability of energy dissipation of PC members could be improved by adding mild steel reinforcement. A lot of experimental and theoretical studies have been carried out on PC members and assemblies since 1965. However, the investigations concerned with dynamic response of PC buildings were reported little except the papers by Spencer(Ref.4), Park(Ref.5) and authors(Ref.2).

In this paper a hysteretic model of PC buildings was proposed, and the nonlinear responses of PC buildings using this model were reported through some case studies carried out on single degree-of-freedom(SDOF) and multi degree-of-freedom(MDOF) systems.

2 MODELING OF RESTORING CHARACTERISTICS OF PRESTRESSED CONCRETE STRUCTURE

The proposed model(PC Model) is shown in Fig. 1. The PC Model was idealized to express the experimentally determined relationship between equivalent viscous damping factor, h_{eq}, and ductility factor, μ.

The idealization is composed of a series of straight lines and is divided into two stages, i.e. before yielding and after yielding.

i) Before yielding
The rule before yielding is shown in Fig. 1(a). Initially on loading before yielding, the path is followed with elastic stiffness, K_1, to the initial positive inelastic point with coordinates(δc, Qc), where δc and Qc are the deformation and load at initial cracking, respectively. For loading beyond this point, the path is followed to a point with the coordinates(δy, Qy), where δy and Qy are the deformation and load at yielding. Before yielding, the stiffness of unloading path starting from the unloading point(δe, Qe) beyond cracking displacement depends on the value of α' and is given, as Eq. 1.

$$Ke' = (Qe - 0.8\alpha' Qc)/(\delta_e - \delta_f) \qquad (1)$$

On unloading a path is followed from the unloading point(δe, Qe) to a point with coordinates(δ_f', $0.8\alpha'$Qc). From a point(δ_f', $0.8\alpha'$Qc) unloading path is followed initial elastic stiffness, K_1, to a point with coordinates(δ_f, 0). Residual deformation, δ_f, is calculated directly from Eqs. 2 and 3,

$$\delta_f = \delta_e - Qe/Ke \qquad (2)$$

$$Ke = K_0 + (1 - \alpha')(K_1 - K_0) \qquad (3)$$

where, K_0 is tangent stiffness at unloading point with the coordinates(δe, Qe). The abscissa at the point(δ_f', $0.8\alpha'Qc$) is then calculated as follows,

$$\delta_f' = \delta_f + 0.8\alpha'Qc/K_1 = (\delta e - Qe/Ke) + (0.8\alpha'Qc/K_1) \quad (4)$$

ii) After yielding
Beyond the yielding point with the coordinates(δy, Qy) a loading path has the stiffness of $\gamma \cdot K_1$. On unloading after yielding, a path is followed with the stiffness, Kp,

$$Kp = \{\alpha'/\mu_p + (1-\alpha')\} Ky' \quad (5)$$

$$Ky' = Ky + (1 - \alpha')(K_1 - Ky) \quad (6)$$

$$\mu_p = \delta p/\delta y \quad (7)$$

where Ky is tangent stiffness at yielding point with the coordinates (δy, Qy), δp is deformation at the unloading point, μ_p is ductility factor at the unloading point. The range of the value of α' is follows.

$$0 \leqq \alpha' \leqq 1 \quad (8)$$

When α' equal to 1, the hysteretic characteristics become origin oriented one and this represents idealized fully prestressed concrete section. When α' equal to 0, the hysteretic characteristics become tri-linear one where the slope of unloading path equal to initial stiffness and this represents the hysteretic characteristics of reinforced concrete section.

In this idealization, hysteretic energy dissipation of the systems increase with decrease of the value of α' which represents the effects of prestressing. The value of α' must have strong relation between prestressing steel ratio, λp, defined by Eq. 9,

$$\lambda p = Ap \cdot fpy/(Ap \cdot fpy + Ar \cdot fry) \quad (9)$$

where Ap and Ar are the sectional area of PC tendon and mild steel reinforcement, fpy and fry are the yield stress of PC tendon and mild steel reinforcement, respectively.

The equivalent viscous damping factors, h_{eq}, calculated by proposed PC Model for various values of α' are shown in Fig. 2. The abscissa shows the drift angle in loading tests. In this calculation, it was assumed that yielding drift angle was 1%.
And in Fig. 2 are also plotted many h_{eq} obtained from the hysteretic loops in the static loading tests on PC beams and PC beam-RC column assemblies. The marks designated by white circle, ○ , show the results obtained from the loading tests on PC beams with λp in the range of 0.32 to 0.61 which correspond to partially prestressed concrete members with a large amount of mild steel reinforcement (designated PR series). The marks designated by black circle, ● , show the values obtained from the same tests on beams with λp in the range of 0.71 to 0.83, which correspond to fully prestressed concrete members (designated FR series).

The experimental results indicate the tendency that the increasing ratio of h_{eq} with increasing of drift angle in PR series is greater than the same ratio in FR series. From this figure, the calculated values with α'=0.5 trace the upper bound of PR series, and the values with α'=0.8 almost coincide with the average value of FR series. In other words, the value of α' for PC and RC structure are found to be roughly 0.5 and 0.8, respectively.

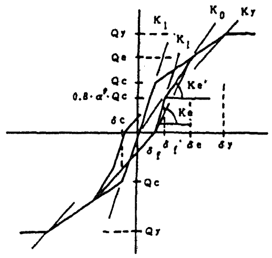
3 NONLINEAR DYNAMIC ANALYSES BY PC MODEL

3.1 Analyses for multi degree-of-freedom systems

The analyses for MDOF systems were carried out on 4, 5 and 9 stories PC buildings (designated Model-A, B and C, respectively), which had been actually designed. In the analyses, the lateral shear capacity and stiffness were determined in accordance with designed values. Parameters were the values of α', that were 0.8, 0.5 and 0.2. The input earthquake motions were El Centro 1940 N-S component and Miyagiken-oki 1978 N-S component, of which maximum acceleration was normalized 300 and 500 gal, respectively.

Figure 3 shows the results of dynamic analyses. In this figure dashed lines show yield interstory drift angle. The responses for 300 gal input acceleration are smaller than yield interstory drift angle except Model-B, those are common results in nonlinear dynamic analyses of PC buildings. It indicates that the maximum response is greatly affected by evaluation of h_{eq} even in range of before yielding. Summarizing the results for α'=0.8, in case of Model-A the maximum response drift angles were appeared in 2nd. story, the values were 12.5×10^{-3} and 29×10^{-3} for 300 and 500 gal input acceleration. In case of Model-B, the maximum responses were 23×10^{-3} and 39×10^{-3} in 5th. story, and in case of Model-C the maximum one were 3×10^{-3} and 19×10^{-3} in 6th. and 4th. story, respectively. If the maximum response drift angle are within above values, it is possible to remain the PC structure under ultimate limit states.

The ratio of response of α'=0.8 to 0.5 in



$$\delta_c < \delta \leq \delta_y$$

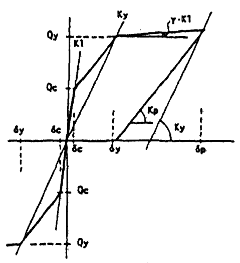
$$K_e = K_0 + (1-\alpha')(K_1 - K_0)$$

$$K_e' = \frac{Q_e - 0.8\cdot\alpha'\cdot Q_c}{\delta_e - \delta_f'}$$

$$\delta_f = \delta_e - \frac{Q_e}{K_e}$$

$$\delta_f' = \delta_f + \frac{0.8\cdot\alpha'\cdot Q_c}{K_1}$$

(a) before yielding



$$\delta_y < \delta$$

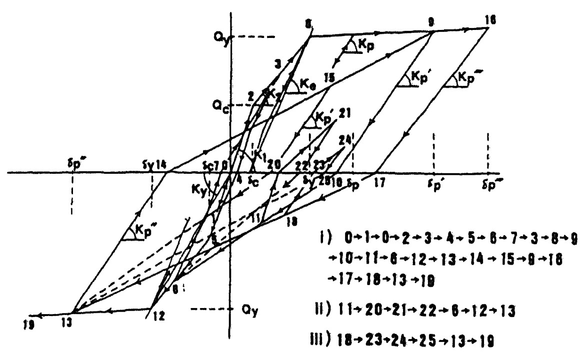
$$K_y' = K_y + (1-\alpha')\cdot(K_1 - K_y)$$

$$K_p = \left\{\frac{\alpha'}{\mu_p} + (1-\alpha')\right\}\cdot K_y'$$

$$\mu_p = \frac{\delta_p}{\delta_y}$$

$$0 < \alpha' \leq 1$$

(b) after yielding



(c) loading sequence

Figure 1. Proposed hysteretic model (PC Model)

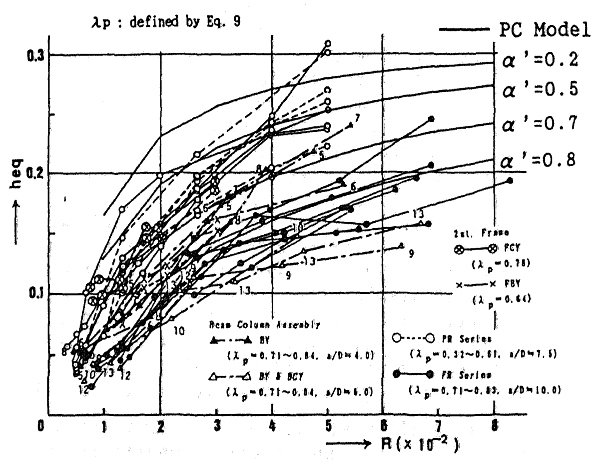
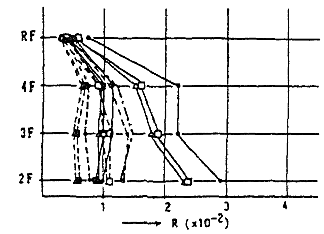
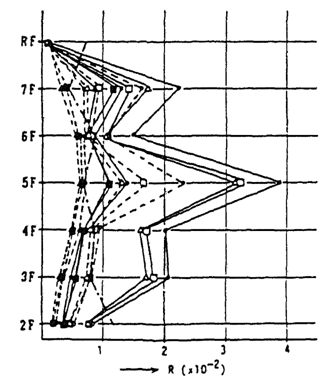


Figure 2. Equivalent viscous damping factor versus drift angle relationship

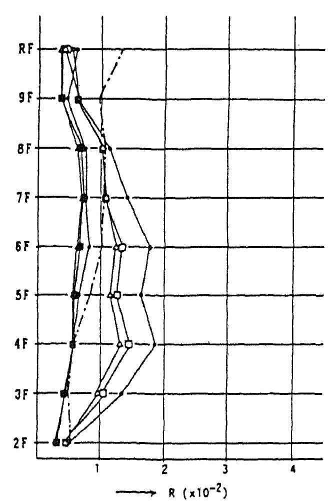
α'	Earthquake	
○ 0.8	Miyagiken-oki	—— 500 gal
□ 0.5		– – – 300 gal
△ 0.2		
● 0.8	El Centro	—·— Yeild interstory drift angle
■ 0.5		
▲ 0.2		



(a) Model-A



(b) Model-B



(c) Model-C

Figure 3. Results of nonlinear dynamic analyses on multi story PC buildings using PC Model

each story, which means comparison of response of PC building to RC building, are shown in Fig. 4. The data are scattered in wide range of 1.0 to 2.0, depending on analytical conditions. The increasing ratio of response are 1.25, 1.20 and 1.17 in average for Model-A, B and C, respectively.

3.2 Response ductility factor by single degree-of-freedom systems

In order to study the relative response of PC and RC SDOF system of which the values of α' in the PC Model were equal to 0.8 and 0.5, respectively, analyses using combination of following parameters were performed,

1)ratio of the base shear coefficient at cracking to that the one at yielding of the system, kc/ky

2)ratio of the base shear coefficient at yielding of the system to the seismic coefficient, where the seismic coefficient is defined as the ratio of the peak acceleration in an earthquake record to acceleration of the gravity, ky/kg

3)ratio of natural period of the system with tangent stiffness at yielding to that with initial stiffness before cracking, Ty/Ti

4)earthquake record, El Centro 1940 N-S component and Hachinohe-Tokachioki 1968 E-W component

All parameters are listed in Table 1.

Response ductility factors for PC and RC SDOF systems are shown in Figs. 5 and 6 against Qy/Qe, where Qy is yield shear force and Qe is elastic response shear force based on initial stiffness. For RC structure ductility factors calculated by Newmark's equal energy concept (shown in Eq. 10) give the upper bound of response ductility factors.

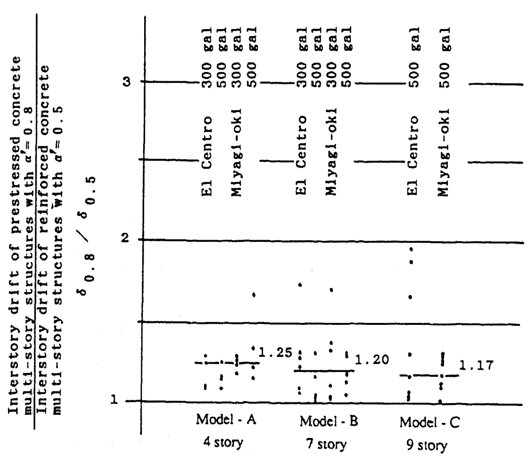


Figure 4. Increased response of PC multi degree-of-freedom system

$$Qy/Qe = 1 / \sqrt{2\mu - 1} \qquad (10)$$

where μ is $\delta/\delta y$.

However for PC SDOF system the upper bound of response ductility factors are somewhat greater than RC system, and the approximation of the upper bound of response ductility factors of PC system is made by Eq. 11,

$$Qy/Qe = \{(2\mu - 1)^{0.15(\alpha')^2}\}/\sqrt{2\mu - 1} \qquad (11)$$

Figure 7 shows the ratios of the response of the system with α'=0.8 and 0.5 versus kg/ky classified by input earthquake record.
Though the increasing ratio are different depending on ky/kg, the responses of PC SDOF systems are greater than RC systems and average values are found to be 1.07 to 1.20.

Table 1. Parameters in nonlinear analyses

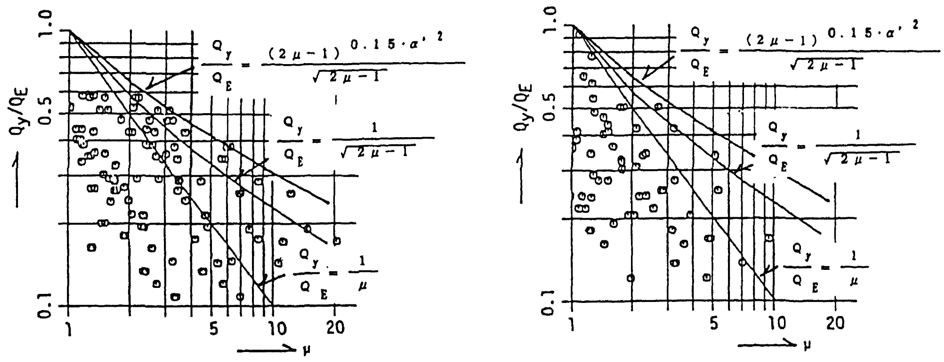
α'	0.5 , 0.8
kc/ky	1/2 , 1/3
ky/kg	0.5 , 0.75 , 1.0 , 1.25 , 1.5
Ty/Ti	2 , $\sqrt{2}$
Ti	0.1 -- 3.0

3.3 Simulation of experimental results on reduced scale 2 storied PC frame by PC Model

Pseudo dynamic tests were carried out on two story 1/3 scaled PC frames consisted of 6 meter PC beams and RC columns. The specimen designated as FBY Frame was designed to make plastic hinges in beam end. The specimen designated as FCY Frame was designed to make plastic hinges in column end. Earthquake response tests using Miyagiken-oki 1985 N-S component were performed for those two specimens . The details of loading tests and the results were reported in Ref.3.

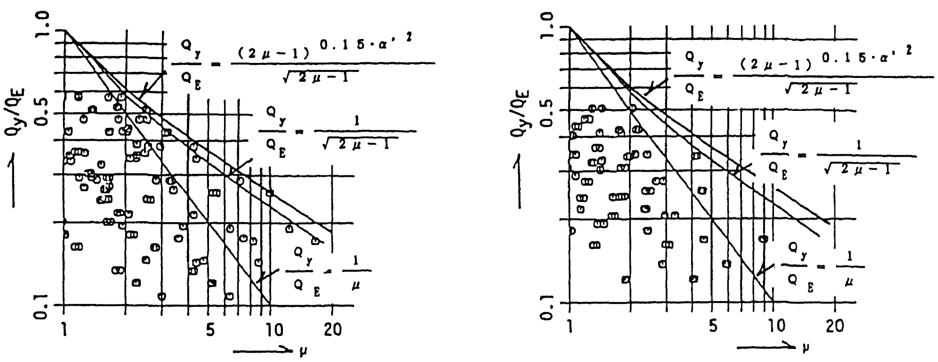
The displacement time histories at roof floor and the relationships of base shear force versus drift angle at roof floor obtained by the pseudo dynamic tests were shown in Fig. 8 and 9, respectively.

Nonlinear analyses using PC Model were carried out to simulate those experimental results. Figure 8 shows the simulation results of displacement time histories comparing with test results. In the analyses, the value of α' was 0.8 for FBY Frame and 0.6 for FCY Frame, respectively, which was determined by the relationships of h_{eq} versus drift angle of test results, shown in Fig. 2. Other analytical conditions were same as the earthquake response tests. And Fig. 9 shows comparison of test and simu-



(a) Hachinohe-Tokachioki 1968 (b) El Centro 1940

Figure 5. Response ductility factor(α'=0.8)



(a) Hachinohe-Tokachioki 1968 (b) El Centro 1940

Figure 6. Response ductility factor(α'=0.5)

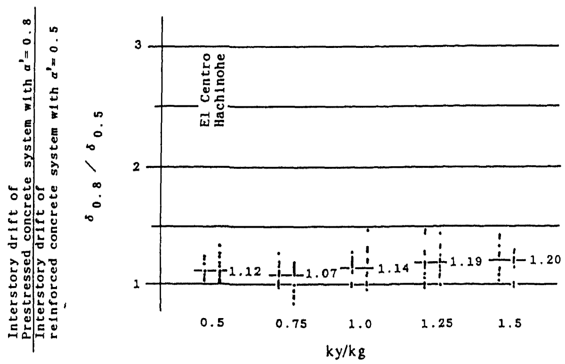
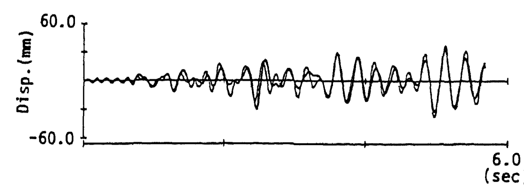
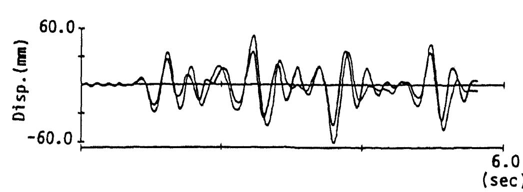


Figure 7. Increased response of PC single degree-of-freedom system



(a) FBY Frame



(b) FCY Frame

Experiment
Analysis(by PC Model)

Figure 8. Comparison of displacement time history between experiments and analyses

lation results in the relationships of base shear force versus drift angle at roof floor. From those figures, the simulations by PC Model can trace adequately the experimental results.

4 CONCLUDING REMARKS

A hysteretic model of PC structures with different yield types or prestressing steel ratio was proposed to simulate the change of the relationship between equivalent viscous damping factors and ductility factors.

Major findings are summarized as follows;

1)In the proposed hysteretic model, equivalent viscous damping factors decrease with increase of value of α', where α' denotes the level of prestressing and the value of α' for RC and PC systems are found to be roughly 0.5 and 0.8, respectively.

2)The upper bound of response ductility factors for RC system are well estimated by Newmark's equal energy concept. For PC system the upper bound of response ductility factors are larger than that of RC system and the approximation of upper bound are made by Eq. 11.

3)The nonlinear dynamic analyses show the 20% increase of response drift angle on an average for PC system compared with RC system.

4)The nonlinear dynamic analyses using PC Model simulate the results of earthquake response tests with reasonable accuracy.

REFERENCES

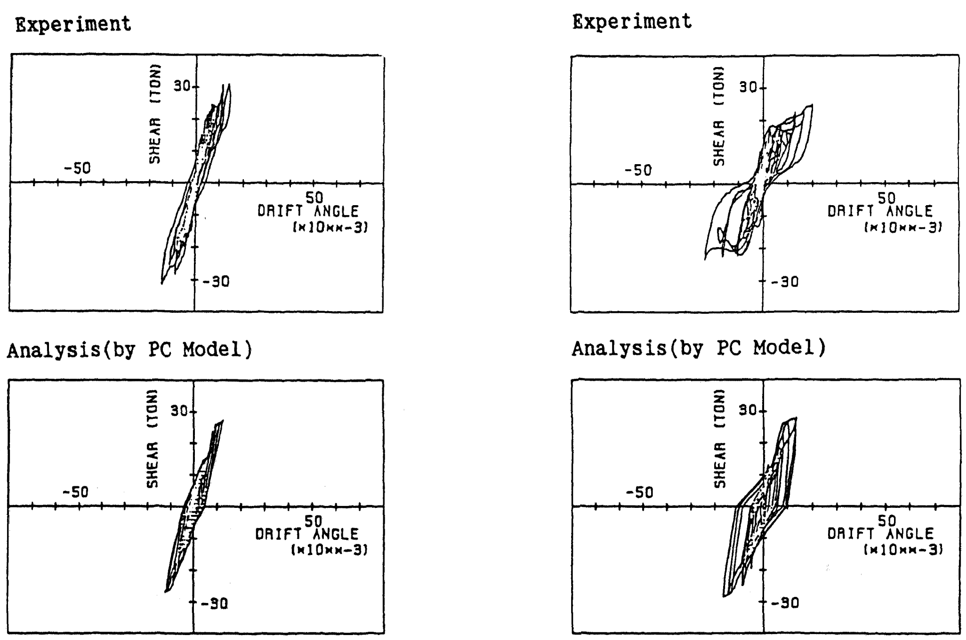
1.Despeyroux, J. 1965. On the use of prestressed concrete in earthquake resistant design. Proc. 3rd. WCEE:IV. 203-215. New Zealand

2.Okamoto, S. & Kato, H. 1991. Dynamic response and hysteretic characteristics of prestressed concrete structures. Research Paper No.135. Build. Res. Inst. Japan

3.Okamoto, S., Yagishita, F. & Kato, H. 1988. Static and pseudo dynamic tests on prestressed concrete model frames. Proc. 9th. WCEE:IV. 89-94. Japan

4.Spencer, R. A. 1969. The nonlinear response of a multistory prestressed concrete structure to earthquake excitation. Proc. 4th. WCEE:A. 4139-4154. Chile

5.Thompson, K. J. & Park, R. 1980. Seismic response of partially prestressed concrete. ASCE Vol.103. ST8:1755-1775



(a) FBY Frame (b) FCY Frame

Figure 9. Comparison of base shear force versus roof drift angle relationships between experiments and analyses