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SEISMIC BEHAVIOUR OF PARTIALLY PRESTRESSED BEAM-COLUMN FRAME ASSEMBLIES

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SUMMARY

In this paper a simple analytical model is calibrated on the basis of results obtained from previous experimental tests carried out to examine the behaviour of partially prestressed concrete systems. The model was calibrated by using an identification technique, within a "Bayesian" context, and used to analyze the performance of a prestressed beam-column assembly under severe earthquake excitations.

INTRODUCTION

Due to a lack of adequate experimental and theoretical research and performance data, prestressed concrete structures are still not widely accepted for use in seismic areas. Despite this there have been significant advances in the study of prestressed concrete structures for use in seismic areas. Some of the most important work in this field has been carried out by Park [1,2]; among his findings he has established that the use of a reasonable level of prestressing improves the hysteretic behaviour of beam-column joints, the energy dissipation of prestressed concrete structures can be increased by the addition of longitudinal and transverse non-prestressed steel, and that partially prestressed concrete, if correctly used, has great potential in seismic resistant construction. The work under discussion in this paper is a continuation of studies carried out at the University of L'Aquila [3,4,5] on the behaviour of partially prestressed concrete structures. An identification technique was applied to an analytical model to identify the cyclic hysteretic behaviour of partially prestressed sections. The model was then used to study the performance of a beam-column assembly, with different levels of prestressing, under severe earthquake excitations.

ANALYTICAL MODEL FOR THE HYSTERETIC BEHAVIOUR

The behaviour, in terms of moment-curvature, of partially prestressed sections can be represented reasonably accurately, in the case under discussion, by combining the responses obtained for prestressed concrete as idealized by Park, and those for reinforced concrete as idealized by the Ramberg-Osgood function. Park's model for prestressed concrete (fig.1a) is made up of three stages; stage 1. the initial elastic range and the post-cracking range; stage 2. crushing occurs in one direction only; stage 3. crushing occurs in both directions. In this case Park's model has been slightly modified as follows: the post-crushing moments do not drop below 30% of the ultimate moment, even when degrading large hysteretic loops. The coordinate M of the "current inelastic point" C_{ip} or C_{in} [1] is not constant, but linearly variable from $0.5M_u$ to $0.3M_u$, in stage 2 or 3, when degrading. The Ramberg-Osgood function, as modified by Menegotto and Pinto [6] has been used for the hysteretic model for reinforced concrete. Fig. 1b shows the shape of the initial loading, unloading and reloading curves. The hysteretic model for the partially prestressed sections (PPM) has been derived by combining the responses, M_p and M_r of the above models at any curvature in such a way as to obtain the total moment

$$M = \alpha_p M_p + (1-\alpha_p) M_r \tag{1}$$

in which $\alpha_p = M_{pu}/M_u$; M_u = the ultimate moment of the partially prestressed concrete section; M_{pu} = the moment of the prestressed steel at ultimate moment capacity.

The basic parameters for defining PPM are: elastic stiffness K_e , post-cracking stiffness K_1, post-crushing stiffness K_2, cracking moment M_{cr}, ultimate moment M_u . All these parameters are collected in the vector x.

EXPERIMENTAL TESTS

High intensity cyclic loading tests were carried out on six simply supported partially prestressed beams. The beams were divided into two groups of three, the concrete sections were prestressed, through a central tendon, at $3N/mm^2$ (Series 1), and at $7N/mm^2$, (Series 2) with α_p equal to .35 and .70 respectively. Fig. 2a shows the overal dimensions of the beams tested, Fig. 2b, shows the loading history used. The moment-curvature relation in the plastic hinge region of each beam was obtained during testing. For further information the reader is referred to [3].

IDENTIFICATION OF THE MODEL

The simplest way to use the PPM for predicting purposes, consists of making a direct evaluation from the experimental moment-curvature cycles of the parameters x. These values are referred to as the "a priori" value x_0 of x. More satisfactory results can be obtained by using a parameter identification technique [5]. However, when the fitting of the experimental data is obtained with parameter values which differ greatly from the "a priori" values a low predicting capacity is obtained. For this reason, an identification technique has been followed which takes into account both the goodness of the fitting of the experimental data and the "a priori" values x_0. This procedure requires the minimization of the following objective function to be resolved by numerical analysis. This function is written as follows:

$$l(x) = \sum_{1}^{N}{}_k \{[z_k - h_k(x)]/Q\}^2 + \beta \sum_{1}^{n}{}_i [p_i (x_i - x_{o,i})]^2 \tag{2}$$

in which z_k represent the N experimental values of the moment-curvature cycle, $h_k(x)$ represent the computed model responses as a function of the n parameters x_i, Q is the maximum experimental moment value; p_i represent coefficients which can vary from 0÷1 and can be used to establish the level of relative importance of the "a priori" estimate x_0 of the parameters; β is factor which expresses the global weight of the "a priori" information in respect of the experimental data. Because of the presence of the second term in (2), the identification procedure can be considered, in a broad sense, as Bayesian.

The minimization of l(x) has been performed by the systematic increase of the β values; the analysis was stopped at the point when a good fitting was obtained between the theoretical and experimental moment-curvature cycles and the parameters were similar in value to the "a priori" values. The adimensional error assumed to check the goodness of the fitting is

$$C = \sqrt{\sum_{1}^{N}{}_k \{[z_k - h_k(x)]/Q\}^2 / N} \tag{3}$$

The p_i coefficients selected for the parameters M_u, M_{cr} , K_e, K_1 and K_2 are 1, 0.7, 0, 0 and 0 respectively. For each of the two series, the most suitable set of identified parameters for the seismic analysis were chosen for β=10. The results of the identification procedure are shown in Table 1. The first line of each section of the table shows the experimental values of the parameters for each series. The first line of results for each beam represents the identified parameters, divided by their own "a priori" value, for the case $\beta=0$, and the second, the identified parameters for the case $\beta=10$. The adimensional error values C are shown in the last column, and the mean values of the identified parameters for the case $\beta=10$ are shown directly below each section of the table. Fig. 3 shows the responses of the PPM to the experimental data of Series 1 and 2. These results are considered as good.

TABLE 1

	SERIES 1						SERIES 2					
	M_{CR} kNm	M_u kNm	K_e kNm^2	K1 kNm^2	K2 kNm^2	C	M_{CR} kNm	M_u kNm	K_e kNm^2	K1 kNm^2	K2 kNm^2	C
Mean Exp. Values	9.70	30.80	2346	1260	-100		15.00	25.50	2830	528	-200	
Beam 1	2.25	0.85	0.54	0.26	0.21	0.119	1.40	0.88	0.63	0.08	1.29	0.128
	1.00	0.97	0.34	0.61	1.19	0.130	1.00	0.98	0.61	0.75	1.18	0.147
Beam 2	1.43	0.83	0.80	0.22	0.05	0.107	1.30	0.83	0.48	0.11	1.61	0.094
	1.00	0.95	0.65	0.96	1.11	0.119	1.00	0.96	0.42	1.03	1.95	0.110
Beam 3	1.88	0.84	0.84	0.16	0.00	0.099	1.40	0.89	0.48	0.11	1.48	0.117
	1.00	0.56	0.56	0.55	0.93	0.114	1.00	0.97	0.48	0.66	1.49	0.132
Mean Id. Values	9.70	29.46	1212	639	-108		15.00	24.72	1424	429	-308	

SEISMIC ANALYSIS

The principal aspects of the seismic behaviour of a partially prestressed frame system has been studied by taking the seismic response of a single degree of freedom system, representing the force-displacement relation ($F - \delta$) of a beam column frame assembly. The simple frame system considered was made up by linking plastic hinges and elastic elements to form a beam-column assembly, Fig. 4, with the prestressed beam framing on both sides at mid-height of the column (interior joint). By assuming a beam sidesway mechanism the post-elastic behaviour was limited to the beam. The PPM was used to relate the beam moments to their curvatures in the plastic hinges. The mean values of the two sets of identified parameters (case $\beta=10$) have been used to characterize the two diverse levels of prestressing in the beam ($\alpha_p=.35$ and $\alpha_p=.70$). Further a reinforced concrete assembly was taken for comparison purposes and Menegotto and Pinto's function was used to represent the behaviour of the plastic hinges. The elastic characteristics of the assemblies chosen were those which closely reproduce the behaviour of typical frame structures.

Assuming a seismic excitation, the equation of motion is as follows:

$$\ddot{y} + \omega_0^2 f(y) = -\omega_0^2 \tilde{a}(t)/\gamma \qquad (4)$$

in which y is the displacement divided by the conventional yielding displacement δ_y, ω_0 the natural frequency, $\tilde{a}(t)$ the acceleration divided by its maximum value a_{max}, $\gamma = F_y/(m \cdot a_{max})$ the design coefficient and f(x) the structural reaction F divided by its maximum plastic value F_y of the beam-column assembly. The conventional yielding displacement δ_y has been defined, on the basis of an energetic criterion, as the displacement, when the two shaded areas are equal, at the intersection of the line from the origin and the horizontal line from the maximum plastic value F_y, Fig. 5.

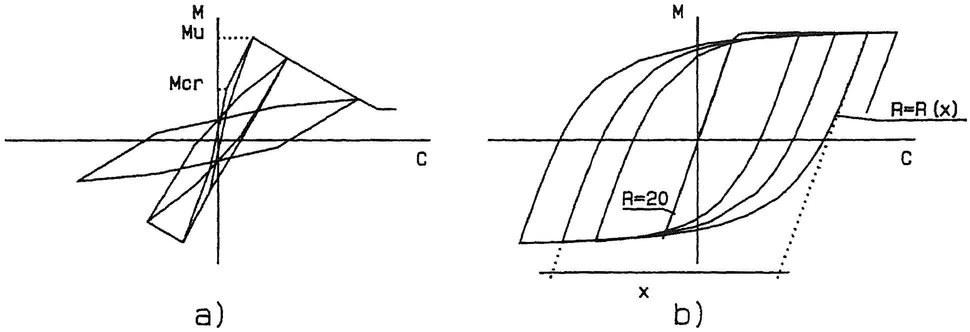

Fig.1

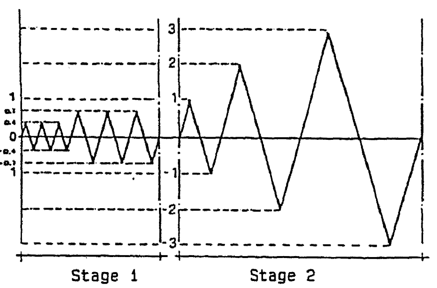

Fig. 2b

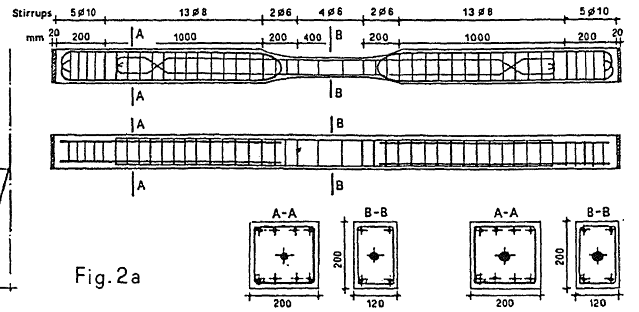

Fig.2a

	Series 1	Series 2
Non prestr. steel:	2+2 Ø14 (A_s =616mm^2)	2+2 Ø8 (A_s =201mm^2)
Prestressed steel:	1 tendon 1/2" dia. (A_{sp} =92.4mm^2)	2 tendons 1/2" dia. (A_{sp} =184.8mm^2)

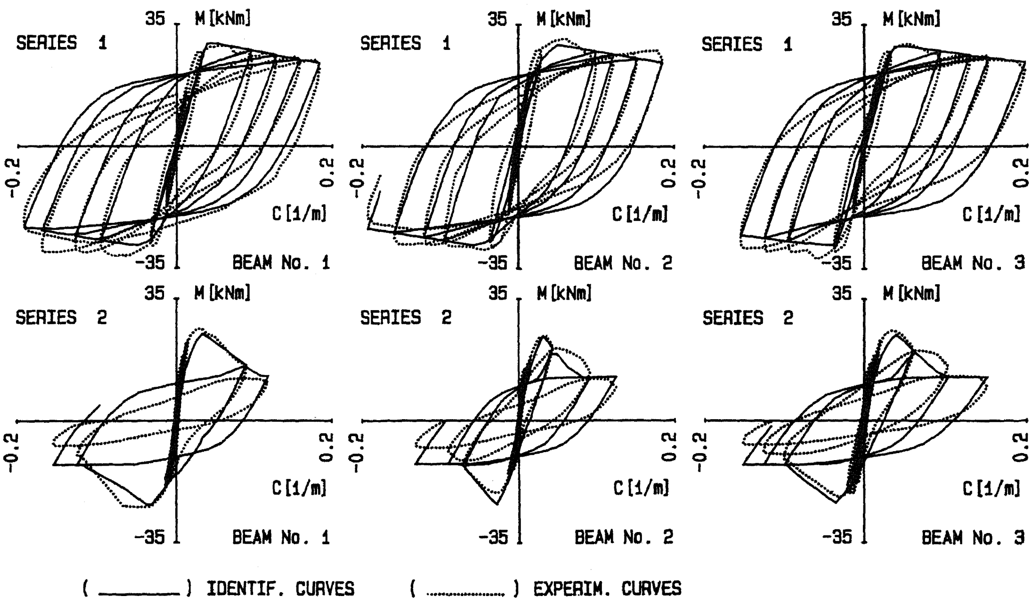

Fig.3

The response spectra of two earthquake accelerograms, Figs. 6a-b, recorded at Sturno and Calitri in Italy. have been computed from eq. (4) by using a step-by-step numerical integration method. The ductility spectra are shown in Figs. 7a-b and 8a-b, with γ=0.5÷1.75; the design spectra are shown in Figs. 9 and 10 with the ductility factor μ=4÷6. As shown in the diagrams the ductility factor demand and the design coefficient, both increase as the level of prestressing is augmented, as a consequence of the decreasing hysteretic energy dissipation capacity, on the other hand, the available ductility decreases in the case of high levels of prestressing and decreases in even higher levels when the prestressing is applied through a central tendon in the beam. This trend is particularly evident in the period range 0.1-0.5 sec, where the dynamic effects are more pronounced. When dealing with large periods the dissipation capacity has less influence on the behaviour of partially prestressed concrete and the trend becomes similar to that of reinforced concrete. Encouraging results have been obtained for the low levels of prestressing, because of the similarity of the design spectra to that of reinforced concrete.

CONCLUSIONS

The simple analytical model for the behaviour of a partially prestressed system subjected to cyclic loading examined is characterized by a limited number of basic parameters having clear physical meaning.

The results obtained from the identification technique demonstrate that the analytical model matches the experimental results, with a reasonable high level of accuracy, and is sufficiently accurate for use in seismic analysis.

The seismic analysis, under severe levels of excitation, of a prestressed beam-column frame assembly shows that when the level of prestressing in the beam increases the ductility factor demand and the design coefficients increase. Good performance can be obtained for low and medium levels of prestressing.

This study does not cover all areas of prestressing concrete, because the results obtained regard only a particular type of prestressing. Further studies should be carried out, in particular on the distribution of prestressed steel within the structural elements.

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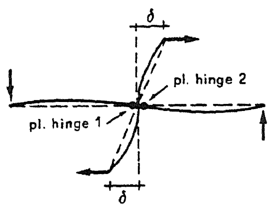

Fig. 4

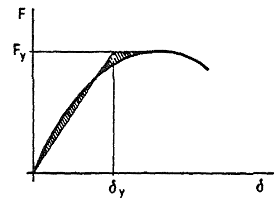

Fig. 5

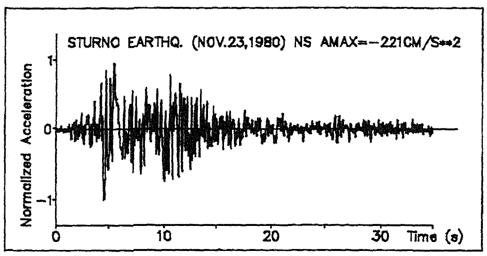

Fig. 6a

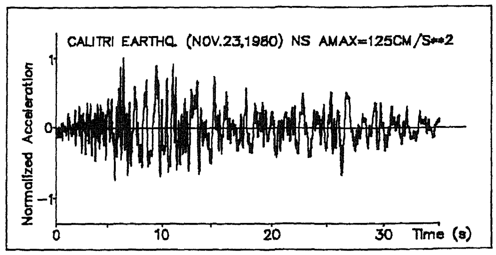

Fig. 6b

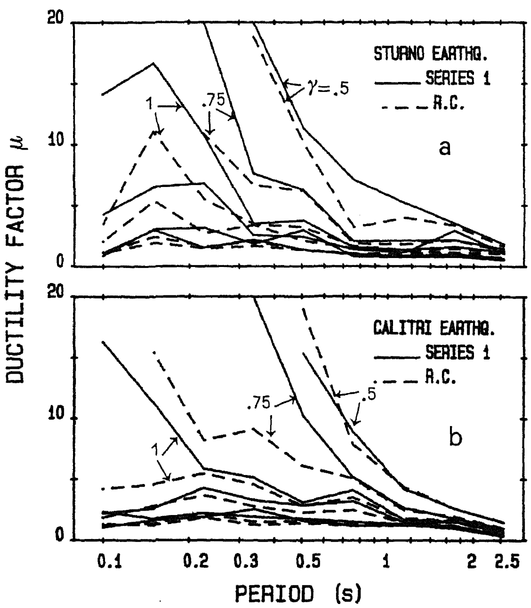

Fig. 7

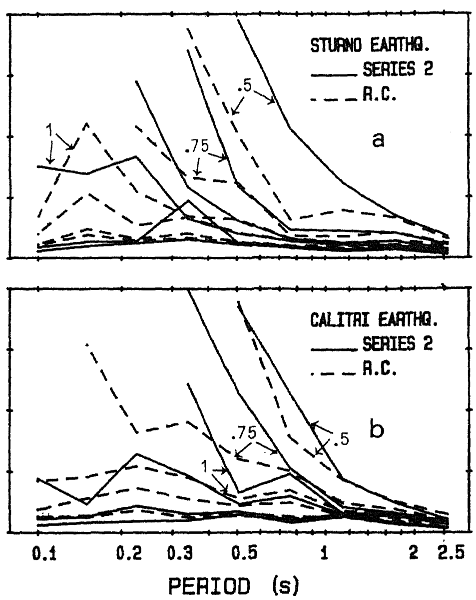

Fig. 8

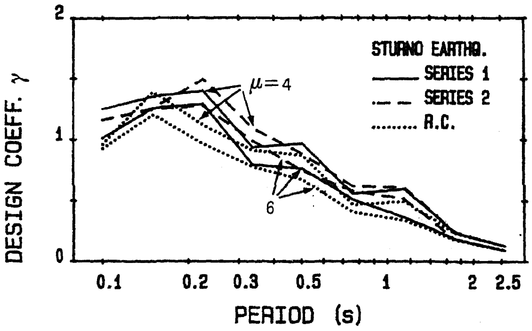

Fig. 9

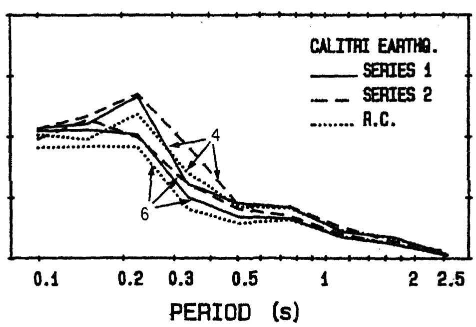

Fig. 10