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CHARACTERISTIC SEISMIC ACCELERATION PARAMETERS AND ITS INFLUENCE ON THE NONLINEAR BEHAVIOR OF STRUCTURES

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ABSTRACT

This paper describes in the first part some strong motion acceleration parameters in the time domain. Further, a classification of the ground motions at a site according to their damage potential is presented. The second part deals with a nonlinear dynamic analysis of a six storey reinforced concrete plane frame. Energy based damage indicators in the form of cross sectional ductility demand are evaluated and correlated with the strong motion characteristics. The attention is focused on earthquake acceleration records in Greece.

KEYWORDS

Acceleration parameter, damage potential, damage indicator, ductility, reinforced concrete.

INTRODUCTION

It is well known that earthquake accelerograms by itself, provide general information only, like peak ground acceleration (PGA) or total seismic duration. For further information extraction, a computer supported elaboration of the accelerograms is necessary. The results of this investigation are characteristic parameters either in the time or in the frequency domain. Some parameters in the time domain are the ARIAS intensity, the HUSID diagram, the strong motion duration and the power $P_{0.9}$. These will be closer defined in the next chapter. On the other hand, some parameters in the frequency domain are the spectral intensities, the response and the Fourier spectra. Additional parameters to the above are presented in the literature (Jennings, 1982).

The observation of building damages during strong motion earthquakes shows that the seismic damage potential is often not well correlated with maximum, one parameter, general specifications like the peak ground acceleration or the total seismic duration. Other, more appropriate descriptors connected with the energy of the motion are better to be used. The above mentioned ARIAS intensities, strong motion duration and power $P_{0.9}$ are some of the energy related descriptors. So, here first some of these sophisticated characteristic seismic parameters are presented and applied on some Greek, Central and North America seismic acceleration records.

After that, the appropriateness of these descriptors to characterise the damage potential of the excitation is shown by carrying out a nonlinear dynamic analysis on a six storey reinforced concrete plane frame system and comparing the ductility demand and supply in critical sections for various excitations. The realistic material behavior is taken into account by the multilinear TAKEDA type law.

DAMAGE POTENTIAL DESCRIPTION

It is known that the destructiveness of a seismic excitation can not always be described by the peak ground acceleration alone, without corresponding spectral data. The damage potential of the seismic excitation correlates better to energy based descriptors (Meskouris *et al.*, 1993; Elenas *et al.*, 1994; Elenas *et al.*, 1995a and 1995b) such as ARIAS intensities (Arias, 1970), energy response spectra and strong motion duration values (Trifunac *et al.*, 1975; Trifunac *et al.*, 1994). The ARIAS intensity is a measure of the energy content of a seismic excitation. The HUSID diagram is the time history of the seismic energy content scaled to the total energy content. The strong motion duration is defined (Trifunac *et al.*, 1975) as time elapsed between 5% and 95% of the HUSID diagram. Finally, the Power $P_{0.9}$ is the ARIAS intensity over the strong motion duration and is a measure of the energy content per time unit of the seismic excitation.

Table 1 shows the PGA, the ARIAS intensities, the time S_{05} at 5% and S_{95} at 95% of the HUSID diagram, the strong motion duration and the Power $P_{0.9}$ of the Zakynthos (1988), Argostoli (1983), San Salvador (1986), Kalamata (1986), scaled Zakynthos, scaled Argostoli, scaled San Salvador, El Centro (1940), Edessa (1990), Pyrgos (1993) and Loma Prieta (1989) earthquake acceleration records. The three scaled records are calibrated to the same PGA as the Kalamata earthquake. Figure 1 shows the HUSID diagrams of some of the examined acceleration records. They are the same for the scaled records respectively.

NONLINEAR DYNAMIC ANALYSIS OF A R.C. FRAME

The six storey reinforced concrete frame system shown in figure 2 has been designed after the recent Greek codes for concrete and aseismic structures. These codes are strongly influenced by the corresponding eurocodes (EC2 and EC8). Then, it has been analysed with material behavior considered as linear and nonlinear one under seismic excitation, according to the records of the Zakynthos, Argostoli, San Salvador, Kalamata, scaled Zakynthos, scaled Argostoli, scaled San Salvador, El Centro, Edessa, Pyrgos and Loma Prieta earthquakes. The modified Takeda/Powell model was taken to describe the nonlinear behavior of reinforced concrete. The computations for the linear and nonlinear dynamic response of the frame, as far as the ductility demand of the beam and column cross sections, were carried out by the program NILDYN (Meskouris *et al.*, 1988). The evaluation of the ductility supply of the cross sections was carried out by the program ZNSQ (Meskouris *et al.*, 1988).

Table 2 shows the maximum linear and nonlinear horizontal displacement response of node 25. As the results in table 2 show, the linear analysis overestimates the displacement response due to disregarding the damping produced by plastic hinges. The original acceleration records of the Zakynthos and Argostoli earthquakes provided only a linear response. Figure 3 shows the curvature ductility demand and supply of the column cross section A-A, provided for all the examined seismic excitations. Here it is shown that the demand depends on the energy content of the seismic motion (ARIAS intensity) in consideration with the strong motion duration expressed by the Power $P_{0.9}$. It is obvious for excitations with the same PGA (Kalamata, scaled Zakynthos, scaled Argostoli and scaled San Salvador). Figure 4 shows the positions of the plastic hinges for some of the examined records. The original Argostoli and Zakynthos excitation provided no hinges. Here it is also shown that the plastic hinge formation depends on the energy content of the seismic motion (ARIAS intensity) interrelated with the strong motion duration. The PGA parameter alone cannot provide the information about the seismic damage potential. Further, in strong seismic excitation the formation of plastic hinges in columns cannot be avoided completely. Finally, as the numerical analysis has shown, the column design in accordance with the Greek aseismic code (capacity design criterion after EC8) leads to a more advantageous behavior than column design based on the Greek code for concrete structures. The latters aseismic requirement is that the sum of flexural strengths of the columns meeting at a joint, under the most unfavourable axial load, be at least equal to 1.15 times the sum of the design flexural strengths of the girders in the same plane framing into the joint. Figure 5 shows the maximum curvature ductility demand and supply of the column cross sections of all storeys, provided for the Kalamata seismic excitation. Here it is shown that plastic hinges can be appeared on the top storey, due to the weaker design demand of the Greek aseismic code.

Table 1. Strong motion characteristics of the acceleration records.

Seismic excitation	PGA [m/s^2]	ARIAS intensity [m^2/s^3]	S_{05} [s]	S_{95} [s]	Strong motion duration [s]	Power $P_{0.9}$ [m^2/s^2]
Zakynthos (1988)	1.670	2.064	3.98	15.9	11.9	0.14
Argostoli (1983)	1.620	2.025	5.90	16.8	10.9	0.18
San Salvador (1986)	3.740	7.865	1.74	5.76	4.02	1.80
Kalamata (1986)	2.679	4.649	2.4	8.6	6.2	0.68
Zakynthos (scaled)	2.679	5.315	3.98	15.9	11.9	0.40
Argostoli (scaled)	2.678	5.541	5.90	16.8	10.9	0.51
San Salvador (scaled)	2.679	4.039	1.74	5.76	4.02	0.80
El Centro (1940)	3.14	7.672	1.43	5.47	4.04	1.70
Edessa (1990)	0.996	1.355	4.70	12.5	7.8	0.15
Pyrgos (1993)	4.45	2.183	2.95	7.11	4.15	0.47
Loma Prieta (1989)	6.17	20.311	2.52	9.36	6.84	2.72

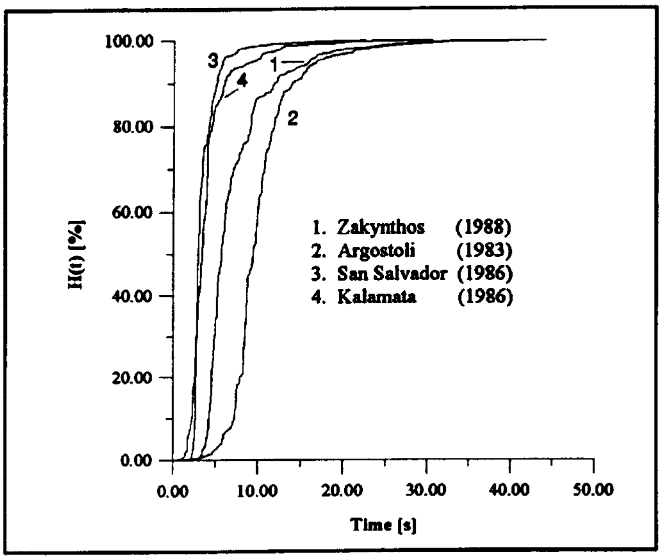

Fig. 1 HUSID diagrams.

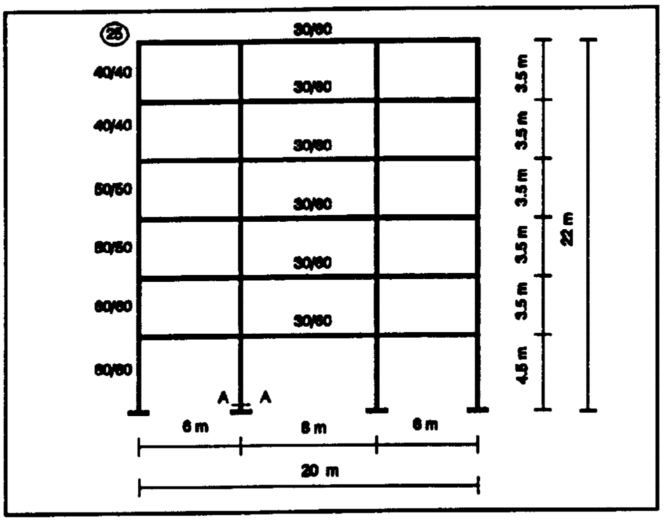

Fig. 2. Reinforced concrete frame system.

Table 2. Maximum horizontal displacement response of node 25.

Earthquake	Linear response [cm]	Nonlinear response [cm]
San Salvador	12.5	11.1
Kalamata	10.4	9.4
Zakynthos (scaled)	4.5	4.5
Argostoli (scaled)	6.7	11.2
San Salvador (scaled)	8.9	8.4
El Centro	13.0	11.3
Edessa	6.4	6.3
Pyrgos	5.0	4.9
Loma Prieta	15.9	14.1

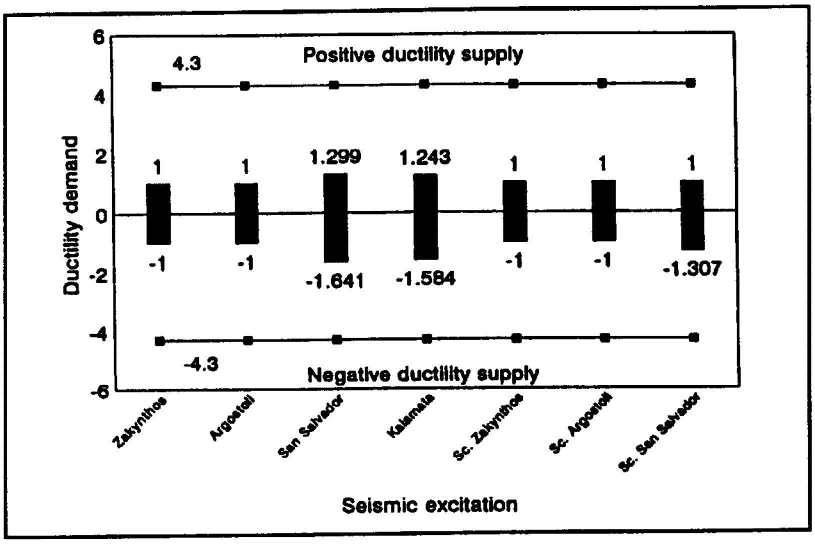

Fig. 3. Curvature ductility demand and supply of cross section A-A.

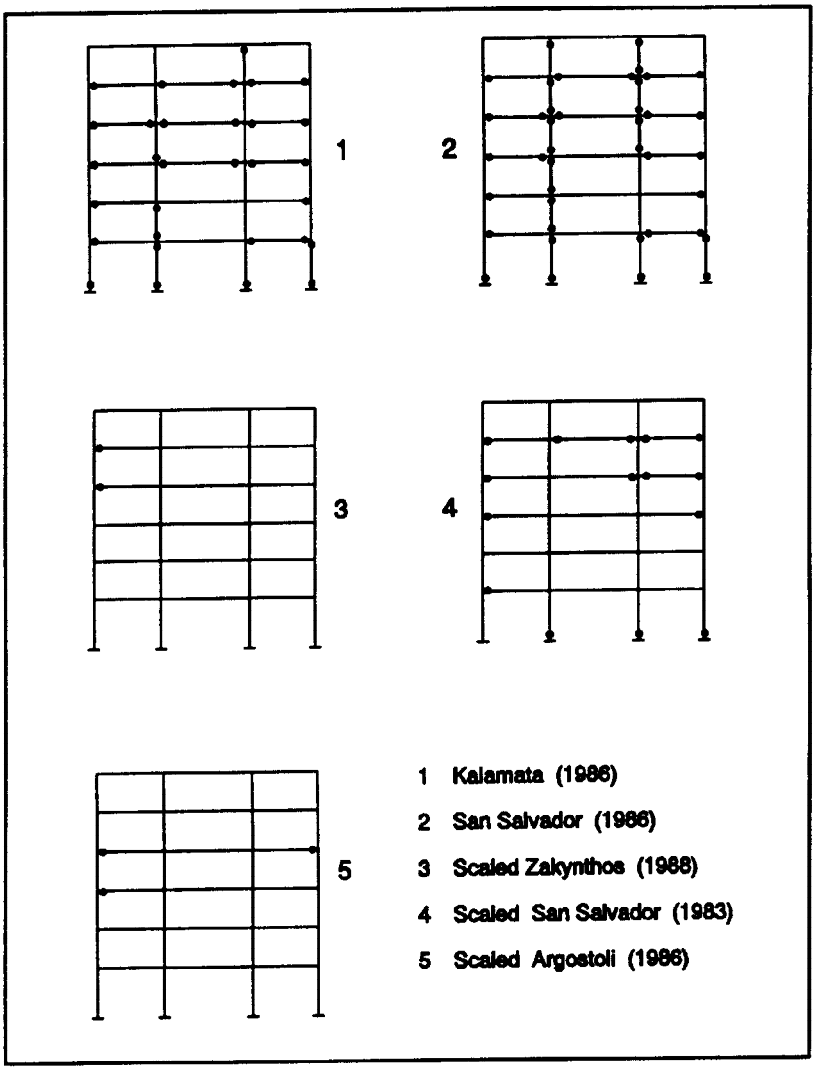

Fig. 4. Plastic hinge positions.

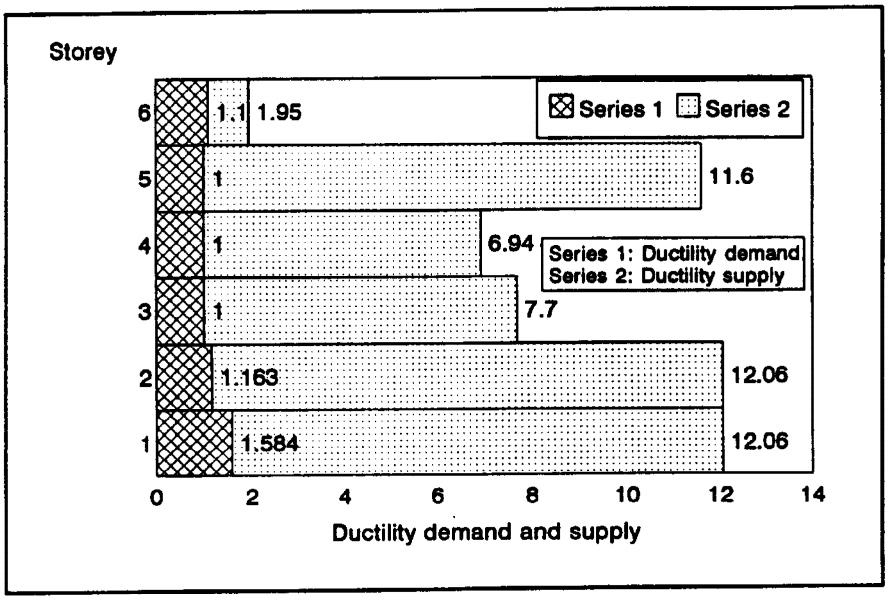

Fig. 5. Maximum column curvature ductility demand and supply. Earthquake of Kalamata (1986).

CONCLUSION

In this paper has been shown that the peak ground acceleration (PGA) by itself is not a satisfactory descriptor of the seismic damage potential. As the numerical example here has shown, the HUSID diagrams as well as the ARIAS intensity interrelated with the strong motion duration expressed by the Power $P_{0.9}$ are better ones (table 1 and figures 1 and 4). On the other hand, in low energy excitations the maximum nonlinear displacement response can be lower than the linear one, due to damping produced by the plastic hinges (table 2). Further, for equal peak ground acceleration the ductility demand correlates to the Power $P_{0.9}$ of the excitation. The latter can be described by the ARIAS intensity, (table 1, figures 2 to 5) over the strong motion duration. Generally, the calibration of a seismic parameter (for example PGA) of different strong motions is a helpful tool to eliminate its influence on other seismic parameters or on the structure response. Finally, as the numerical example has shown, the aseismic codes based on the capacity design criterion of the columns, like the new Greek aseismic code and EC8, leads to satisfactory structural behavior, even the plastic hinge formation in columns cannot be completely avoided (figure 4).

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