OPTIMUM DESIGN OF THE STRUCTURAL MEMBERS DUE TO GROUND MOTION

by

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SYNOPSIS

Optimum distribution of the dynamic characteristics of the structural members with elasto-plastic joints is discussed, which minimize the deviation of the root mean square responses of ductility ratios of column and girder joints due to random excitation. Then, the elastic and inelastic responses of the structures subjected to some kinds of earthquakes are computed in order to clarify the difference between elastic and inelastic responses considering plastic flow of axial deformation. The results of the analyses are presented as the aseismic design data of structures composed of strong girders or those composed of weak girders.

INTRODUCTION

During past 10 years, the inelastic earthquake response of multi-story framed structure considering local hysteretic characteristics of structural material or members has been studied by various investigators. Most of analyses, however, are performed by using the inelastic relation between bending moment and curvature or rotation without considering the dynamic effect of axial-flexural interaction except for a few studies (1,2). It is very important to grasp the inelastic behavior of column members, usually subjected to the gravity load as well as the three dimensional earthquake motions.

In this paper, we discuss mainly on the distribution of local ductility ratio responses such as column and girder joints of plane structures, considering the effect of axial force and axial deformation with reference to the studies on the optimum seismic design based on spring-mass systems (3,4).

PROCEDURE OF ANALYSIS

For details of the fundamental equations and analysis techniques, reference should be made to the author's previous paper (1). However, the most significant assumptions used in the modelling of structures are given in the following:

The model structure considered is 7 stories high with a uniform story height, bay width of twice the story height and fixed at the base as shown in Fig.1. It is assumed that the ratio of the length of an elasto-plastic joint to the length of its member $\Delta L/L = 1/20$, the ratio of the story height to the depth of the member $\bar{L}/\bar{H} = 20/3$ and the mass distribution is uniform. Three important parameters are introduced to specify the optimum distribution of the dynamic characteristics: stiffness and elastic limit bending moment ratio of the i-th story column to the base column defined as $k_i = b_i = 1-\lambda\{\frac{i-1}{6}\}^{\nu}$, the elastic limit strength ratio $\bar{\beta}$ of the girder joints to the column joints belonging to a girder-column assemblage, which means that shear type structure corresponds to the case $\bar{\beta}>>1$ and bending type to the case $\bar{\beta}<1$, and the ratio of the elastic limit bending moment to the product of elastic limit axial force by member depth defined as $\mu = My/NyH$,

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which represents a measure of axial flexibility of the column member. The case where $\mu = 0$ represents negligence of axial deformation, while the case $\mu = 1/2$ corresponds to the idealized sandwitch section. Nondimensional axial force of the 1st story due to gravity load can be determined by using the base shear coefficient s and the elastic limit base shear force as, for instance, $n = \{\frac{1+\bar{\beta}}{s}\frac{\bar{H}}{L}\}/\{1+\frac{1+\bar{\beta}}{s}\frac{\bar{H}}{L}\mu\}$ under the assumption of the square yielding condition. The case where $s = 0$ represents negligence of the gravity load and, for instance, $s = 0.2$ means the case considering it. In the latter case, P-Δ effect, axial plastic flow and initial bending moment due to gravity load are considered dynamically in elasto-plastic analysis.

From the above distributions representing the dynamic characteristics of the model structures, the nondimensional fundamental frequencies are obtained as shown in Fig.2 which indicates that the effect of P-Δ on the frequency is large for bending type structures and the effect of axial deformation is large for shear type structures.

The accelerograms used to compute are classified by three groups. The first group denoted by Nos.1-5 is the group of artificial earthquakes (AE) generated by the random pulses with 10 sec. duration, zero mean and unit standard deviation. The 2nd group Nos.6-10 is that of the filtered earthquakes (FE) having $f = 2Hz$, $h = 0.6$ and the last group Nos.11-13 is that of the recorded earthquakes (RE), i.e., El Centro NS, EW and Vernon S82°E. All of these are normalized so as to have the unit maximum amplitude and zero final velocity. Fig.3 shows the average spectra of artificial earthquakes and filtered earthquakes and the spectrum of El Centro NS earthquake. The frequency parameter ψ is selected in this paper as 30, 60, 90 and 120, which correspond to the dimensional fundamental periods $T_1 = 0.3$, 0.6, 0.9 and 1.2 sec. for the duration time $T_d = 10$ sec. respectively. Strength parameter α is defined as $\alpha = \frac{A}{g}\frac{2(1+\bar{\beta})}{7s}(1-n)$, where A is the maximum acceleration of the excitation. The value α considered here is 0.2, 0.3, 0.4 and 0.5 corresponding to the above-mentioned value of ψ to have the same average ductility ratio in elastic response. For example, the maximum acceleration of excitation is equal to 0.13g supposing $T_1 = 0.6$ sec., $s = 0.2$, $\bar{\beta} = 0.6$, $\alpha = 0.3$ and $n = 0$.

NUMERICAL RESULTS

Figs.4 through 7 show the deviation of the root mean square elastic responses of relative displacements, ductility ratios of column joints and girder joints due to random excitation. Minimum deviation of column responses as well as that of displacement responses in the case $\bar{\beta} = 50$, $\mu = 0$ occurs at $\lambda = 0.7$, which is close to the case of shear type structure. Optimum distribution for the column ductility may not be so affected by $\bar{\beta}$ and μ than that for the relative displacement. But, it is influenced by the higher mode vibration and the axial-flexural interaction, that is to say, the former affects on the upper part of structure and the latter on the lower part as shown in Fig. 5(c),(d). Comparing Fig. 5(a) with Fig. 6 in detail, it seems that the optimum value of λ is larger for column and smaller for girder when $\bar{\beta}$ becomes smaller. This fact suggests that the column strength is to be increased in the lower part. Fig. 7 shows that it seems to be most advantageous to design such a structure as to be equal strength of girder and column in elastic stage but may be dangerous because of the interaction effect in plastic stage. The distribution of elastic responses for the case when $\lambda = 0.7$ are shown in Fig. 8, where the ductility ratio of column joint is almost uniform except the column joint at the base for $\bar{\beta} \lesssim 1.0$, but the column response in the lower part when considering the axial-flexural interaction (slender line in Fig. 8) becomes occasionally greater than girder response. Ductility ratio responses of the relative displacements defined in reference to the elastic limit strength of neighboring joints are naturally uniform.

Fig. 9 and 10 show the distribution of the undamped linear responses due to earthquakes. When subjected to AE Quake, the distribution has the same tendency as afore-mentioned stationary random responses. However, that due to FE and RE Quakes is very different depending on the frequency parameter ψ . The deviations of column ductility of the structures emphasized its fundamental mode such as $\psi = 30, 60$ take its minimum when $\lambda = 0.7$–0.8 but those influenced by higher modes such as $\psi = 90, 120$ have the minimum value at $\lambda = 0.6$–0.7 . The latter fact is the similar conclusion as reference (3) for high rise structure. In Fig. 10, the distribution of the column ductility ratios normalized by its value of the bottom joint of the 1st story is presented, which shows that the response of structure with longer period than the predominant period of the excitation is large in the upper part.

Last two figures and table show the nonstationary nonlinear responses of ductility ratios due to FE Quake of the elasto-plastic joints with and without considering gravity load. In spite of the uniformity of linear response, elasto-plastic responses in each joints in the case of weak column variate as shown in Fig. 11. However, the responses of the structure with weak girder seem to be uniform in girder ductility, and the responses of column joints remain almost uniformly in elastic region except for the column joint at the base. In the case of $\psi = 90$, $\bar{\beta} = 0.6$, column ductility response is comparable to girder ductility response due to higher mode vibration. Considering the gravity load, distribution of local responses become smooth due to the time difference entering into plastic region between left and right joints, but the column joints in the lower part have a tendency to be in plastic region because of interaction effect. Fig. 12 and table show that the ductility ratio of axial deformation at the base joint is accumulated every plastic behavior in the case of $s = 0.2$ where axial force due to gravity load is about 25 % of elastic limit axial force. Axial deformation becomes considerably small when s is equal to 0.3 in the case that axial force is about 15 % of yielding force.

CONCLUSION

From the above results, the following remarks will be recommended;

1. It does not garrantee about the safety of each member of a structure with weak girder, to control only the distribution of the responses of the relative displacements, uniformly.
2. When the fundamental frequency of structures is in the range of the predominant frequency of the ground motion, it is desirable for controling the column responses in elastic or slighty plastic region whereas the girder response in considerably plastic that the strength distribution parameter λ is about 0.7 and the strength ratio of girder to column $\bar{\beta}$ is 0.6 .
3. In the case where the frequencies of higher modes of vibration is in the predominant frequency region of the excitation, the values of λ and $\bar{\beta}$ may be selected so as to be smaller than in the above case, because the column responses are disposed to behave plastically.
4. As the plastic deformation of the base column of structures designed by small base shear coefficient tends to be accumulated, the capacity of plastic deformation should be examined and the suitable selection of base shear coefficient will be recommended to control the plastic behavior in axial deformation.

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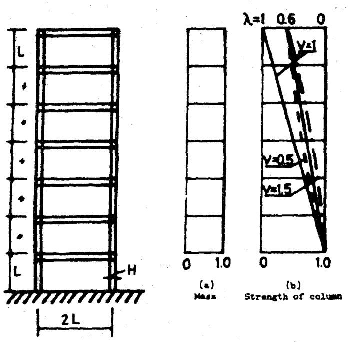


Fig.1 Model structure

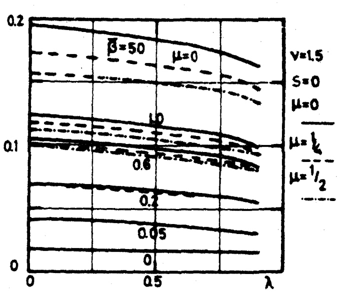


Fig.2 Nondimensional fundamental frequency

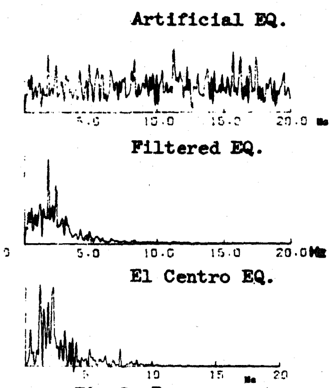


Fig.3 Power spectrum

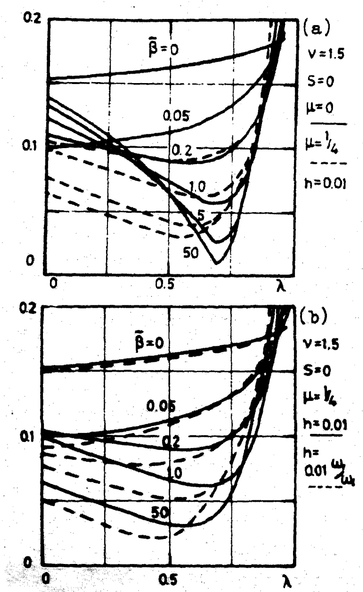


Fig.4 Deviation of relative displacement

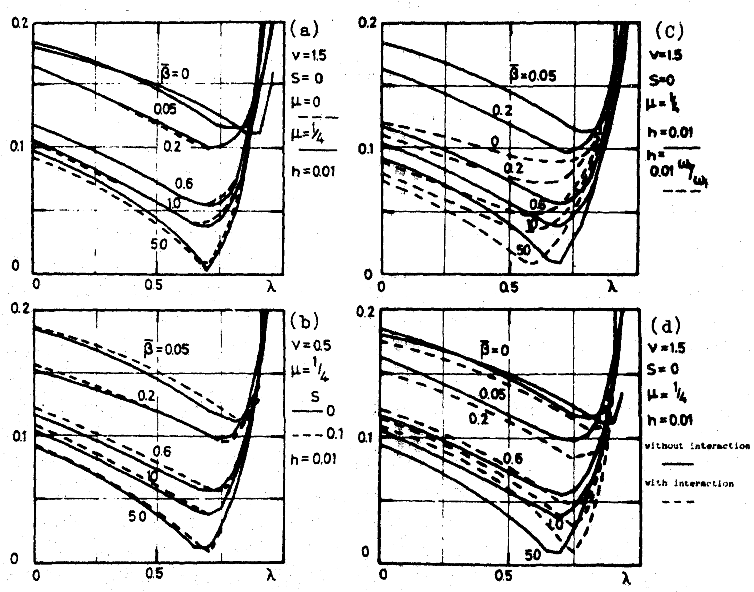


Fig.5 Deviation of ductility ratio of column joints

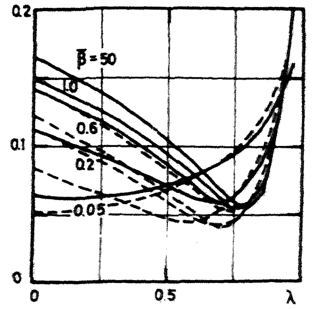


Fig.6 Deviation of ductility ratio of girder joints

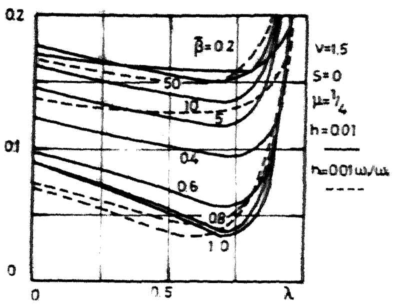


Fig.7 Deviation of ductility ratio of all joints

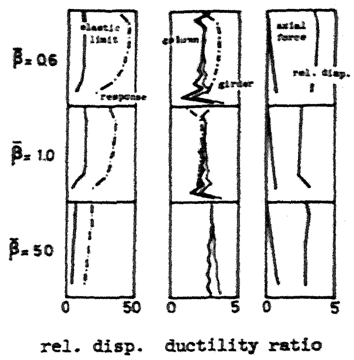


Fig.8 Distribution of responses : s=0, λ=0.7, ν=1.5, μ=0.25

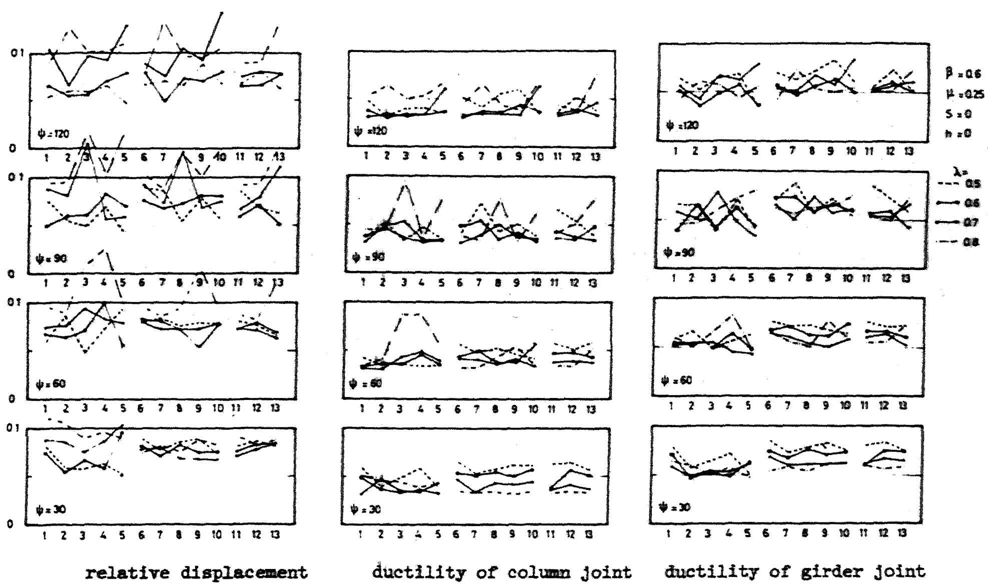


Fig.9 Deviation of nonstationary undumped linear responses of model structure : β=0.6, s=0, λ=0.7, ν=1.5, μ=0.25

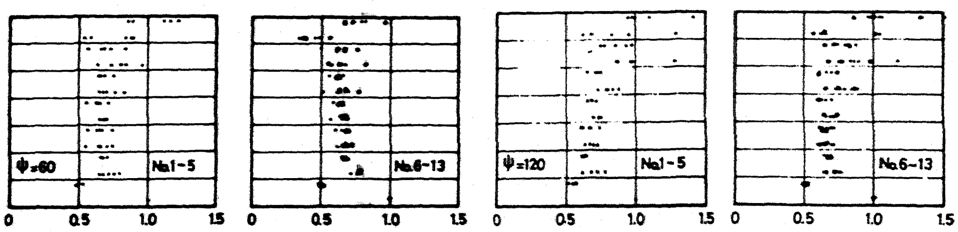
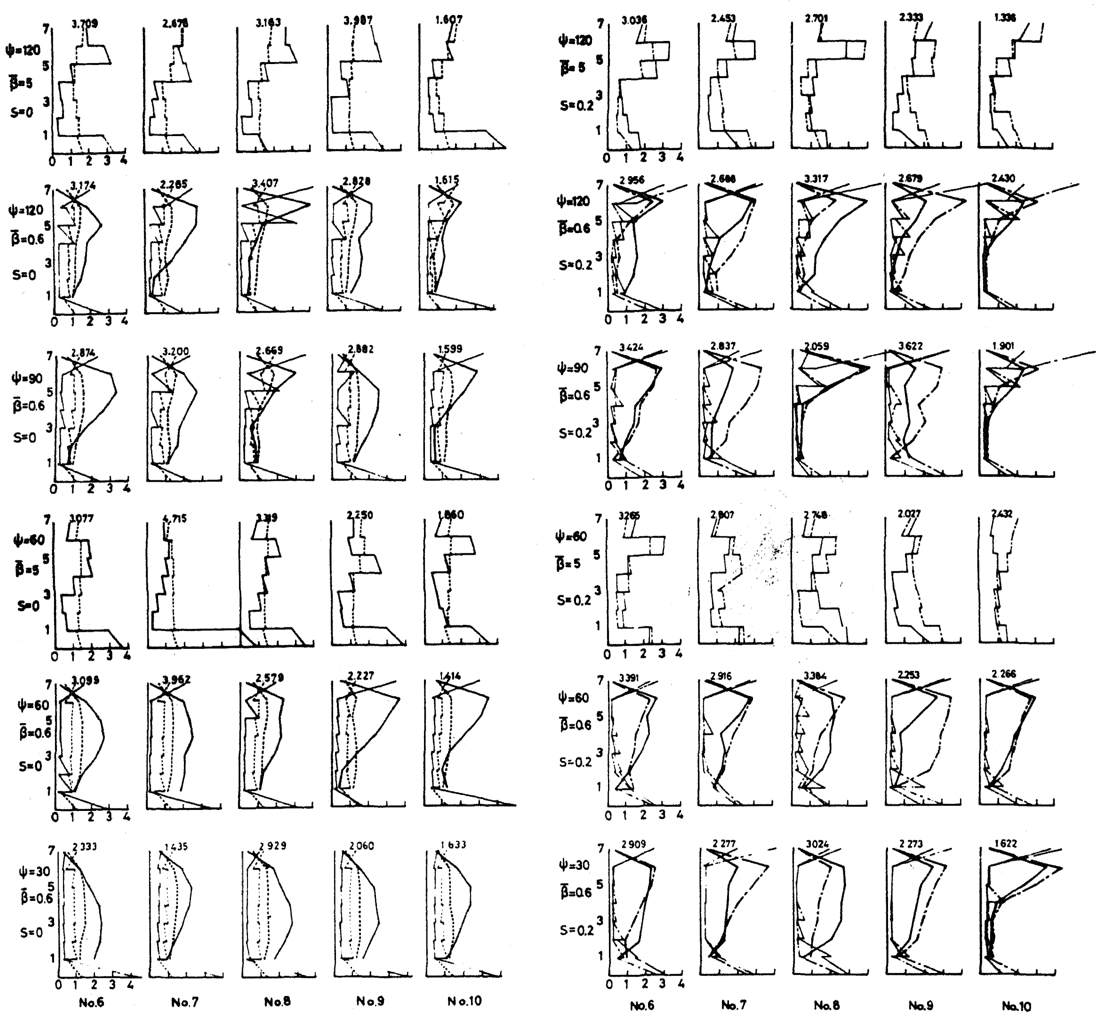


Fig.10 Distribution of elastic ductility response of column joints : β=0.6, s=0, λ=0.7, ν=1.5, μ=0.25



—————— elasto-plastic

- - - - - elastic

—————— left joint of sym. frame

— — — — right joint of sym. frame

Fig.11 Distribution of maximum ductility ratio of relative rotation of joints normalized normalized by the average value shown on the top of each figure : thick line - girder , slender line - column

Table I Absolute maximum ductility ratios of relative rotation θ and axial deformation δ of the column joint at the base

excitation		No. 6		No. 7		No. 8		No. 9		No.10	
		θ	δ	θ	δ	θ	δ	θ	δ	θ	δ
S= 0	ψ= 30	10.29	9.31	5.57	4.76	10.57	9.55	8.55	7.63	6.42	5.56
	ψ= 60	9.30	8.38	14.43	13.27	8.06	7.17	3.69	2.93	6.94	6.08
	ψ= 90	7.09	6.20	8.07	7.15	8.11	7.16	9.98	8.97	5.69	4.78
	ψ=120	8.30	7.32	4.77	3.95	7.66	6.74	8.49	7.50	6.71	5.88
S=0.2	ψ= 30	9.52	99.49	7.02	54.82	9.11	76.84	5.27	52.57	2.00	5.87
	ψ= 60	9.37	113.98	6.38	101.18	7.29	68.65	4.37	17.36	4.80	38.38
	ψ= 90	6.40	39.50	8.86	61.67	2.02	4.49	9.38	67.64	3.06	7.74
	ψ=120	8.81	32.70	6.13	36.14	8.67	50.25	4.48	24.09	3.65	11.21

$\bar{\beta}$=0.6, λ=0.7, ν=1.5, μ=0.25

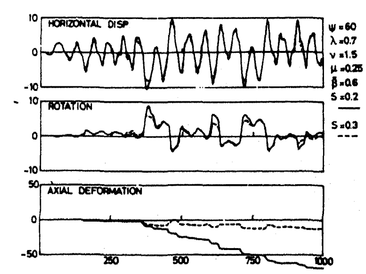


Fig.12 Time histories of the displacement and the ductility for rotation and axial deformation of the bottom joint of the 1st floor