NATIONAL CENTER FOR EARTHQUAKE
ENGINEERING RESEARCH
State University of New York at Buffalo

STATISTICAL EVALUATION OF RESPONSE MODIFICATION FACTORS FOR REINFORCED CONCRETE STRUCTURES

by

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NOTICE

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PREFACE

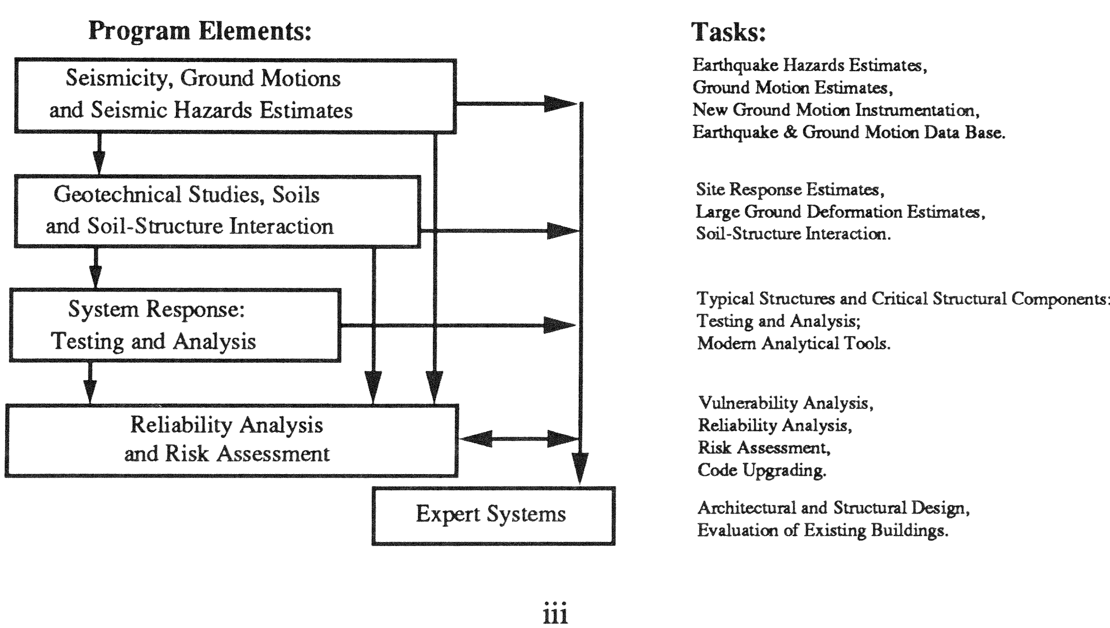
The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures and lifelines that are found in zones of moderate to high seismicity throughout the United States.

NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

This technical report pertains to Program 1, Existing and New Structures, and more specifically to reliability analysis and risk assessment.

The long term goal of research in Existing and New Structures is to develop seismic hazard mitigation procedures through rational probabilistic risk assessment for damage or collapse of structures, mainly existing buildings, in regions of moderate to high seismicity. This work relies on improved definitions of seismicity and site response, experimental and analytical evaluations of systems response, and more accurate assessment of risk factors. This technology will be incorporated in expert systems tools and improved code formats for existing and new structures. Methods of retrofit will also be developed. When this work is completed, it should be possible to characterize and quantify societal impact of seismic risk in various geographical regions and large municipalities. Toward this goal, the program has been divided into five components, as shown in the figure below:



Reliability analysis and risk assessment research constitutes one of the important areas of Existing and New Structures. Current research addresses, among others, the following issues:

1. Code issues - Development of a probabilistic procedure to determine load and resistance factors. Load Resistance Factor Design (LRFD) includes the investigation of wind vs. seismic issues, and of estimating design seismic loads for areas of moderate to high seismicity.
2. Response modification factors - Evaluation of RMFs for buildings and bridges which combine the effect of shear and bending.
3. Seismic damage - Development of damage estimation procedures which include a global and local damage index, and damage control by design; and development of computer codes for identification of the degree of building damage and automated damage-based design procedures.
4. Seismic reliability analysis of building structures - Development of procedures to evaluate the seismic safety of buildings which includes limit states corresponding to serviceability and collapse.
5. Retrofit procedures and restoration strategies.
6. Risk assessment and societal impact.

Research projects concerned with reliability analysis and risk assessment are carried out to provide practical tools for engineers to assess seismic risk to structures for the ultimate purpose of mitigating societal impact.

This report summarizes a study of the response modification factor R, which is used in design codes to reduce the linear force levels; thus this work relates both to the systems response area and to code and risk analysis. Extensive analyses of twelve stick models, representing reinforced concrete structures, were analyzed for 90 artificial ground motions. Statistical analysis of the results of linear and nonlinear analyses showed that the R values given in codes are too high and should depend on the ductility factor and on the period ratio (the relative values of the initial structure period and the dominant ground motion period). This study indicates that the R values must be reexamined and that it will be necessary to study other types of structural models, such as concrete frames with shear deformations and progressive hinge formations, to see whether the conclusions derived for the stick model remain valid.

ABSTRACT

This report presents a statistical evaluation of the response modification factor for reinforced concrete structures. The response modification factor R is defined as the ratio of the absolute maximum linear elastic base shear to the absolute maximum nonlinear base shear of a structure subject to the same earthquake accelerogram. Twelve structural models with various dynamic characteristics are first constructed. Next, 90 synthetic earthquakes are generated from three power spectra representing different soil conditions. Then, the nonlinear and corresponding linear time history analyses are performed to produce structural response data. On the basis of these data, an empirical formula for the response modification factor is established from a multivariate nonlinear regression analysis. The empirical formula describing the mean value of R factor is a function of the maximum ductility ratio, the viscous damping ratio and the earthquake-structure period ratio. In addition, variation of R factor in terms of the maximum ductility ratio is also established from the multivariate nonlinear regression analysis. The empirical formula is demonstrated using two structures. In addition, comparison of the proposed formula with Newmark's formulas is also made. From the empirical formula, the response modification factors recommended for the design of reinforced concrete structures are also presented. The authors believe that most of the R factors specified in the current NEHRP provisions are too large and unconservative. Thus, the specification of more reasonable R factors in the seismic design provisions is warranted.

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SECTION 1
INTRODUCTION

The current seismic design criteria for building structures allow structures to undergo inelastic deformations under a specified design earthquake. The effect of inelastic deformation on the design base shear, which is reduced from elastic force level, is included in some building codes by a response modification factor. For example, the response modification factor R_w is employed in the 1988 Uniform Building Code (UBC) [1] and the response modification factor R is used in the NEHRP Recommended Provisions [2]. The difference between R and R_w is due to the prescribed design force level. The design force specified in the NEHRP Provisions is at the significant yield level; while the design force prescribed in 1988 UBC is at the allowable stress level. In these codes, however, a constant value of the response modification factor is assigned to each type of structure depending on the construction material and the seismic resisting system. It has been recognized that the response modification factor is affected by many variables such as ductility level and viscous damping [3,4]. Thus, a constant R value specified in building codes for each type of structure may be oversimplified.

Several studies have been conducted to establish empirical formulas for constructing the nonlinear response spectrum from an elastic response spectrum [3-6]. These formulas can be used to establish the response modification factor. However, these formulas were derived on the basis of single-degree-of-freedom (SDF) systems. Since most structures are multi-degree-of-freedom (MDF) system, the application of these formulas for the response modification factor is questionable. Thus, there is a need to establish a practical and reliable formula for the response modification factor.

This report presents a statistical evaluation of the response modification factors for reinforced concrete structures which include frame and shear wall structures. In this study, the response modification factor R is defined as the ratio of the absolute maximum linear elastic base shear to the absolute maximum nonlinear base shear of a structure subject to the same earthquake accelerogram. To generate structural response data, twelve structural models are first constructed from a set of parameters defining the dynamic characteristics of structures. Then, ninety synthetic earthquakes are generated from three power spectra representing different soil conditions. A hysteretic model with stiffness degrading and pinching effect is utilized to describe nonlinear behavior of structure. The nonlinear and corresponding linear time history analyses of each structure subject to earthquakes are carried out to generate response data. Then, a multivariate nonlinear regression analysis is performed to derive an empirical formula for R factor in terms of pertinent parameters such as ductility ratio, viscous damping ratio, etc. The empirical formula is demonstrated using two structures. In addition, comparison of the proposed formula with Newmark's formulas is also made. From the empirical formula, the response modification factors for the design of reinforced concrete structures are also recommended.

SECTION 2
SEISMIC ANALYSIS OF STRUCTURE

In this study, the structure is represented by a multi-degree-of-freedom stick model fixed at the base. The stick model consists of concentrated masses connected by beam elements. Each mass has one degree of freedom, i.e., the horizontal displacement in the direction of earthquakes. The equations of motion for such an MDF system subject to a horizontal earthquake acceleration are

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + \{F_s\} = -[M]\{I\}\ a_g \tag{2.1}$$

where, $[M]$ = mass matrix; $[C]$ = damping matrix; $\{I\}$ = identity vector; $\{X\}$ = nodal displacement vector relative to the fixed base; $\{F_s\}$ = restoring force vector; and a_g = earthquake acceleration. The damping matrix $[C]$ is taken as the Rayleigh damping matrix, which is the combination of the mass matrix $[M]$ and the initial stiffness $[K_e]$ of the structure.

$$[C] = a_0[M] + a_1[K_e] \tag{2.2}$$

where

$$a_0 = \frac{2\zeta\omega_1\omega_2}{\omega_1 + \omega_2}$$

$$a_1 = \frac{2\zeta}{\omega_1 + \omega_2} \tag{2.3}$$

in which ω_1 and ω_2 are the first two natural frequencies of the structure and ζ is the damping ratio for these two modes.

The structure may behave nonlinearly under severe earthquakes. In this study, the hysteretic relationship between restoring shear force Q and inter-story displacement U is described by the modified Takeda model [7]. This model has a bilinear skeleton curve and includes both stiffness degrading and pinching effect. As shown in figure 2-1, the modified Takeda model is governed by the following five rules:

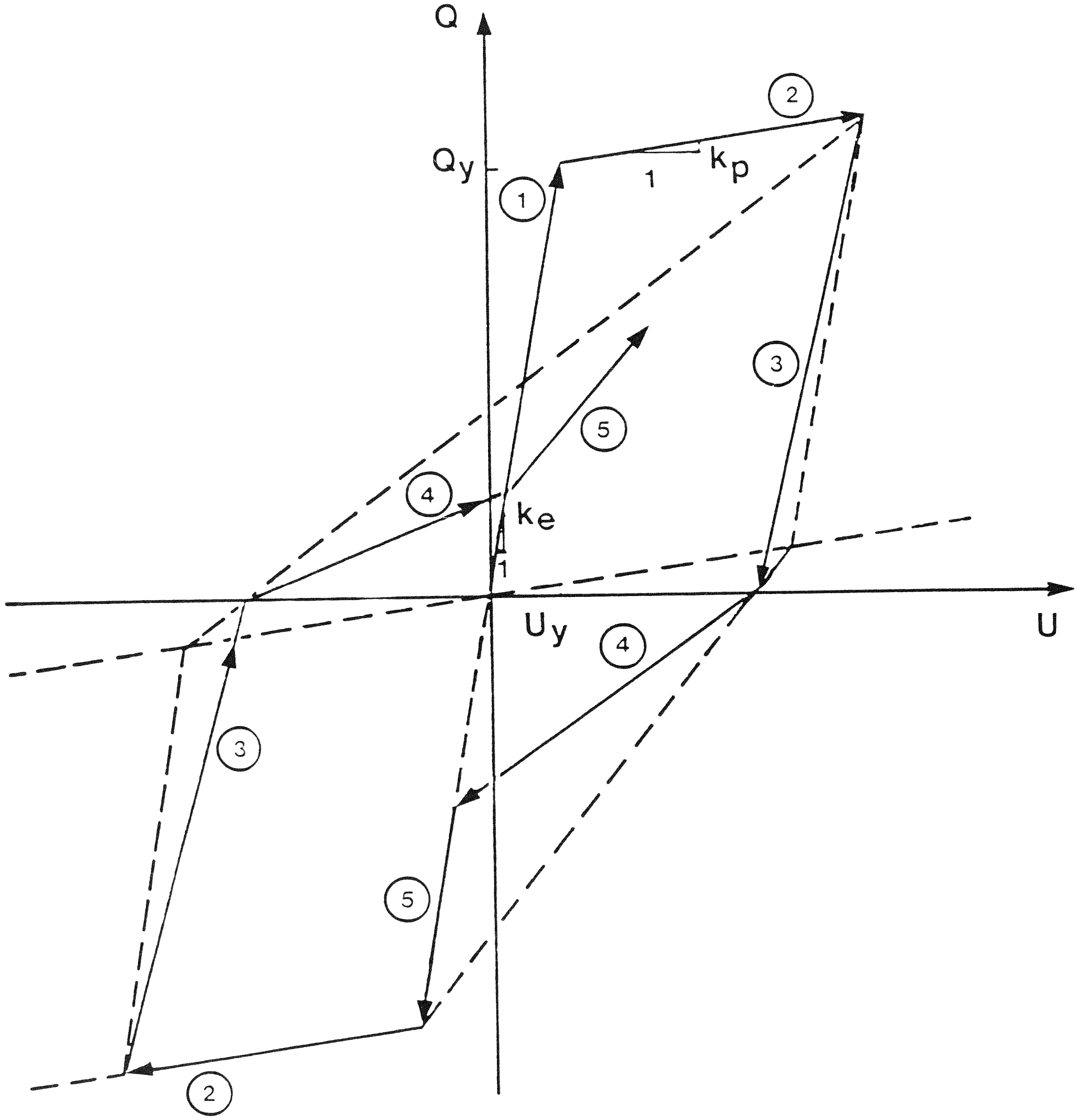

FIGURE 2-1 Hysteretic Diagram

1. Elastic loading and unloading with initial stiffness.
2. Inelastic loading with post-yielding stiffness.
3. Inelastic unloading with degrading stiffness.
4. Inelastic pinched reloading.
5. Peak oriented inelastic reloading.

These five rules result in five possible paths in the hysteretic diagram as identified in figure 2-1 by corresponding numbers in circles. Ref. 7 presents the detailed description of the hysteretic rules. The restoring force vector $\{F_s\}$ in Eq. (2.1) can be derived based on these hysteretic rules.

For a given earthquake time history, the Newmark's beta method with beta equal 1/4 is used to perform step-by-step integration of equations of motion to obtain nonlinear and linear responses of the structure.

SECTION 3
STRUCTURAL MODELS

Twelve structural models as shown in table 3-I are constructed in this study to represent low-rise to mid high-rise reinforced concrete structures. These structures are generated from the combination of number of stories, fundamental period and viscous damping ratio. Stick models A and B are 4-story structure with fundamental period of 0.3 second and 0.6 second respectively, while models C and D are 10-story structure with fundamental period 0.9 second and 1.2 seconds, respectively. The structure with shorter period implies a shear wall structure, while the structure with longer period represents a frame structure.

In each stick model, story mass m and story height are assumed to be uniform for all stories. The fundamental period of a structure is a function of mass and initial stiffness. In this study, the story mass is set equal to 1.0 kip-sec^2/in, while the initial stiffnesses of beam elements are adjusted to achieve the prescribed fundamental period. The initial stiffness is determined with the aid of story yielding strength. For the i-th story, the story yielding strength Q_{yi} is taken as twice the story shear Q_i, which is determined from the requirements of ANSI A58.1 standard [8]. Evaluation of Q_{yi} is shown in Appendix A. In computing initial stiffness, it is assumed that the yielding displacement U_y is identical for all stories in each stick model. This implies that Q_{yi}/k_{ei} is constant for all stories. Thus, initial stiffness k_{ei} of the i-th story can be expressed as

$$k_{ei} = k\left(\frac{Q_{yi}}{Q_{y1}}\right) \tag{3.1}$$

where k is the initial stiffness of the first story and can be determined by

$$k = \left(\frac{T_s'}{T_s}\right)^2 m \tag{3.2}$$

in which m is the story mass; T_s is the prescribed fundamental period of the structure as shown in table 3-I and T_s' is the fundamental period of the structure obtained from the eigenvalue analysis with k equal to unity. Once k is computed, the initial stiffness of other stories can be determined from Eq. (3.1). Furthermore, the yielding displacement of any story in a stick model is

TABLE 3-I Structural Parameters

Structure	Stick Model	Number of Stories	Fundamental Period (sec.)	Viscous Damping Ratio (%)
1	A	4	0.3	3
2	A	4	0.3	5
3	A	4	0.3	7
4	B	4	0.6	3
5	B	4	0.6	5
6	B	4	0.6	7
7	C	10	0.9	3
8	C	10	0.9	5
9	C	10	0.9	7
10	D	10	1.2	3
11	D	10	1.2	5
12	D	10	1.2	7

$$U_y = \frac{Q_{y1}}{k} \tag{3.3}$$

Tables 3-II through 3-V summarize the physical properties of four stick models.

The modified Takeda hysteretic model is utilized in this study to describe the nonlinear behavior of beam elements. For the i-th beam element, the model is characterized by four parameters: the yielding strength Q_{yi}, the initial stiffness k_{ei}, the post-yielding slope factor α_{si}, and pinching factor α_{pi}. Q_{yi} and k_{ei} are shown in tables 3-II to 3-V. The post-yielding slope factor α_{si} is chosen to be 0.03 for all beam elements, which is considered as a typical value for reinforced concrete structures [9]. The pinching factor of 0.3 has been suggested to be an appropriate value for low-rise structures [10]. Thus, this value is adopted for all elements of stick models A and B (4-story structure). It is envisioned that models C and D (10-story structure) are dominated by the flexural behavior and the pinching effect is less significant; therefore the pinching factor is set to be 1.0.

It is well known that damping values vary over a wide range and depend on factors such as the structural material and the stress level during excitation. Considerable judgement is usually involved in selecting appropriate damping values for use in dynamic analysis. The damping ratio ranging from 2 to 10 percent has been recommended for reinforced concrete structures [11]. In this study, the damping ratios of 3, 5 and 7 percent are selected for each structural model.

TABLE 3-II Physical Properties of Stick Model A

Story Number	Mass m (kip-s^2/in)	Initial Stiffness k_e (kips/in)	Story Yielding Strength Q_y (kips)	Story Yielding Displacement U_y (in.)
4	1.0	1755	86.5	0.0493
3	1.0	3071	151.4	0.0493
2	1.0	3948	194.6	0.0493
1	1.0	4386	216.2	0.0493

TABLE 3-III Physical Properties of Stick Model B

Story Number	Mass m (kip-s^2/in)	Initial Stiffness k_e (kips/in)	Story Yielding Strength Q_y (kips)	Story Yielding Displacement U_y (in.)
4	1.0	439	63.7	0.145
3	1.0	768	111.4	0.145
2	1.0	987	143.1	0.145
1	1.0	1097	159.1	0.145

TABLE 3-IV Physical Properties of Stick Model C

Story Number	Mass m (kip-s^2/in)	Initial Stiffness k_e (kips/in)	Story Yielding Strength Q_y (kips)	Story Yielding Displacement U_y (in.)
10	1.0	612	75.9	0.124
9	1.0	1012	125.5	0.124
8	1.0	1369	169.8	0.124
7	1.0	1683	208.7	0.124
6	1.0	1950	241.8	0.124
5	1.0	2173	269.5	0.124
4	1.0	2350	291.4	0.124
3	1.0	2484	308.0	0.124
2	1.0	2574	319.2	0.124
1	1.0	2620	324.9	0.124

TABLE 3-V Physical Properties of Stick Model D

Story Number	Mass m (kip-s^2/in)	Initial Stiffness k_e (kips/in)	Story Yielding Strength Q_y (kips)	Story Yielding Displacement U_y (in.)
10	1.0	366	70.6	0.193
9	1.0	584	112.7	0.193
8	1.0	778	150.2	0.193
7	1.0	949	183.2	0.193
6	1.0	1095	211.3	0.193
5	1.0	1217	234.9	0.193
4	1.0	1315	253.8	0.193
3	1.0	1387	267.7	0.193
2	1.0	1435	277.0	0.193
1	1.0	1461	282.0	0.193

SECTION 4
EARTHQUAKE MOTION

In many engineering applications, recorded ground motion accelerograms are commonly used to represent earthquakes that may be expected at a site. However, this approach has some drawbacks: (1) there is a scarcity of strong motion records in some regions, for example, the eastern United States, and (2) it does not grasp uncertainty in future earthquakes nor properly reflect the local site condition. To avoid these shortcomings, the use of synthetic earthquake time histories to represent ground motion is an appropriate alternative. Synthetic earthquakes may be generated by the following approaches: (1) modify amplitudes and frequencies of recorded accelerograms; (2) develop compatibly from a specified response spectrum; and (3) generate from an appropriate power spectrum. The power spectrum approach is utilized in this study.

The synthetic earthquake time history $a_g(t)$ is generalized from the product of a specified peak ground acceleration A_p and the normalized nonstationary time history $a_m(t)$

$$a_g(t) = A_p \times a_m(t) \tag{4.1}$$

The normalized nonstationary time history $a_m(t)$ is obtained by applying an envelope function $f(t)$ to a stationary time history $a_s(t)$, and then normalized by the absolute maximum of the time history a_{max}.

$$a_m(t) = \frac{a_s(t)f(t)}{a_{max}} \tag{4.2}$$

The stationary acceleration time history $a_s(t)$ is simulated by the following expression [12].

$$a_s(t) = \sqrt{2}\sum_{k=1}^{N_f} \sqrt{S_g(\omega_k)\Delta\omega} cos(\omega_k t + \phi_k) \tag{4.3}$$

where $S_g(\omega)$ = one-sided earthquake power spectrum; N_f = number of frequency intervals; $\Delta\omega = \omega_u / N_f$ with ω_u as cutoff frequency; $\omega_k = k\Delta\omega$, and ϕ_k = k-th random phase angle which is uniformly distributed between 0 and 2π.

The earthquake power spectrum used in this study is a Kanai-Tajimi (K-T) power spectrum [13].

$$S_g(\omega) = S_0 \frac{1 + 4\zeta_g^2(\frac{\omega}{\omega_g})^2}{\left[1 - (\frac{\omega}{\omega_g})^2\right]^2 + 4\zeta_g^2(\frac{\omega}{\omega_g})^2} \tag{4.4}$$

where S_0 is the amplitude of the spectrum and is related to the peak ground acceleration (PGA) [14]; ω_g and ζ_g are the dominant ground frequency and the critical damping, respectively, which depend on the site soil condition. In this study, the power spectra corresponding to three soil conditions are used and the parameters for these three power spectra are tabulated in table 4-I [15]. The strong motion duration d_E is also included in the table. Figure 4-1 shows the three power spectra with PGA = 0.15 g. From each power spectrum, 10 normalized nonstationary time histories are generated. Thus, for a specified PGA level, 30 normalized earthquake accelerograms are produced. It is noted that 30 different sets of random phase angles are used to generate these time histories. Three levels of PGA, i.e., 0.1 g, 0.15 g and 0.2 g are chosen for this study. Therefore, a total of 90 earthquake accelerograms is produced for time history analysis of structures. Figure 4-2 shows a sample of synthetic earthquakes.

TABLE 4-I Earthquake Parameters

Earthquake	PGA (g)	Soil Type	ω_g (rad/sec)	ζ_g	d_E (sec)
1-10	0.1				
11-20	0.15	Rock	8π	0.6	10
21-30	0.2				
31-40	0.1				
41-50	0.15	Deep	5π	0.6	15
51-60	0.2	Cohesionless			
61-70	0.1				
71-80	0.15	Soft	2.4π	0.85	20
81-90	0.2				

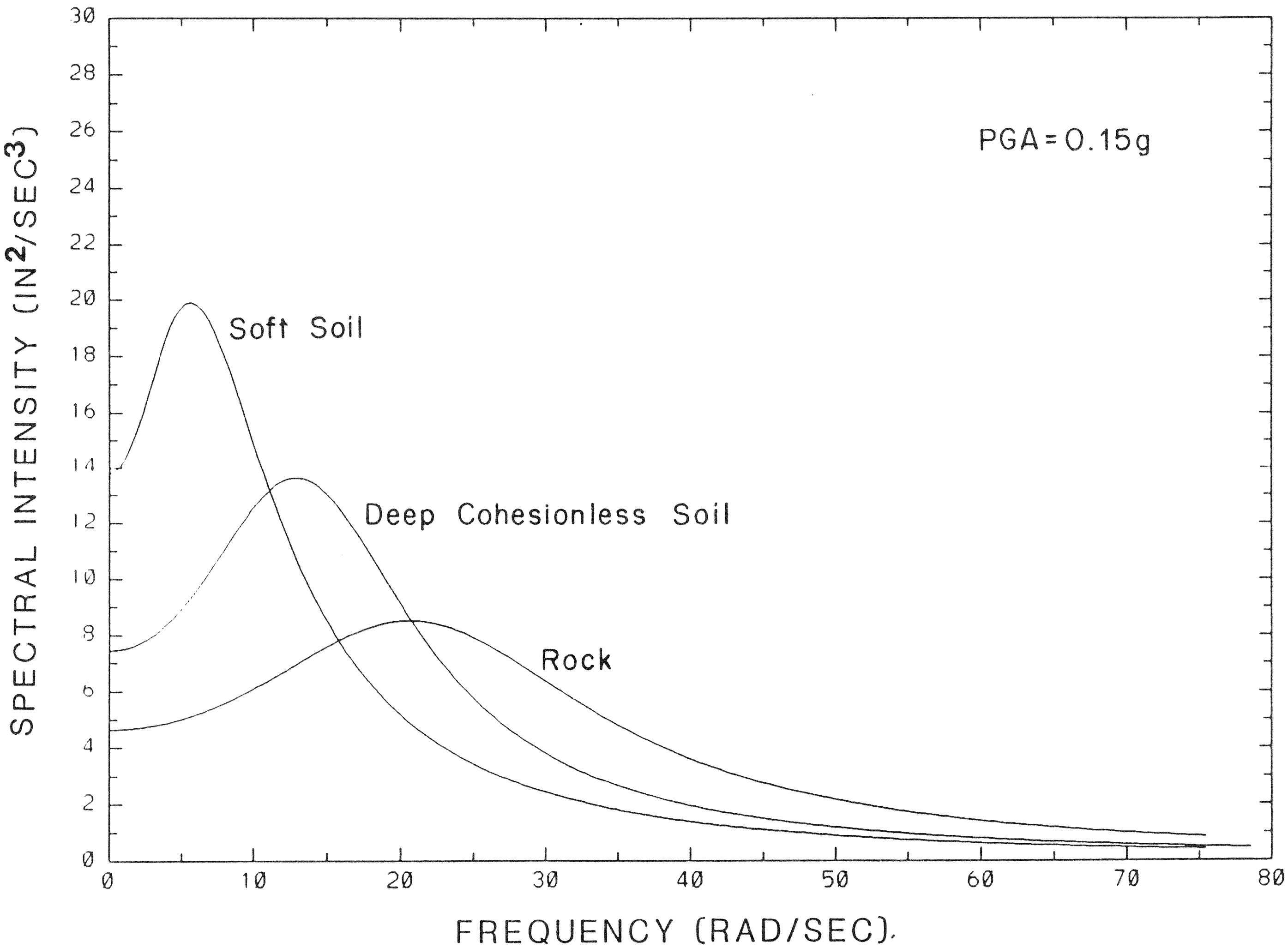

FIGURE 4-1 Power Spectra with Different Soil Conditions

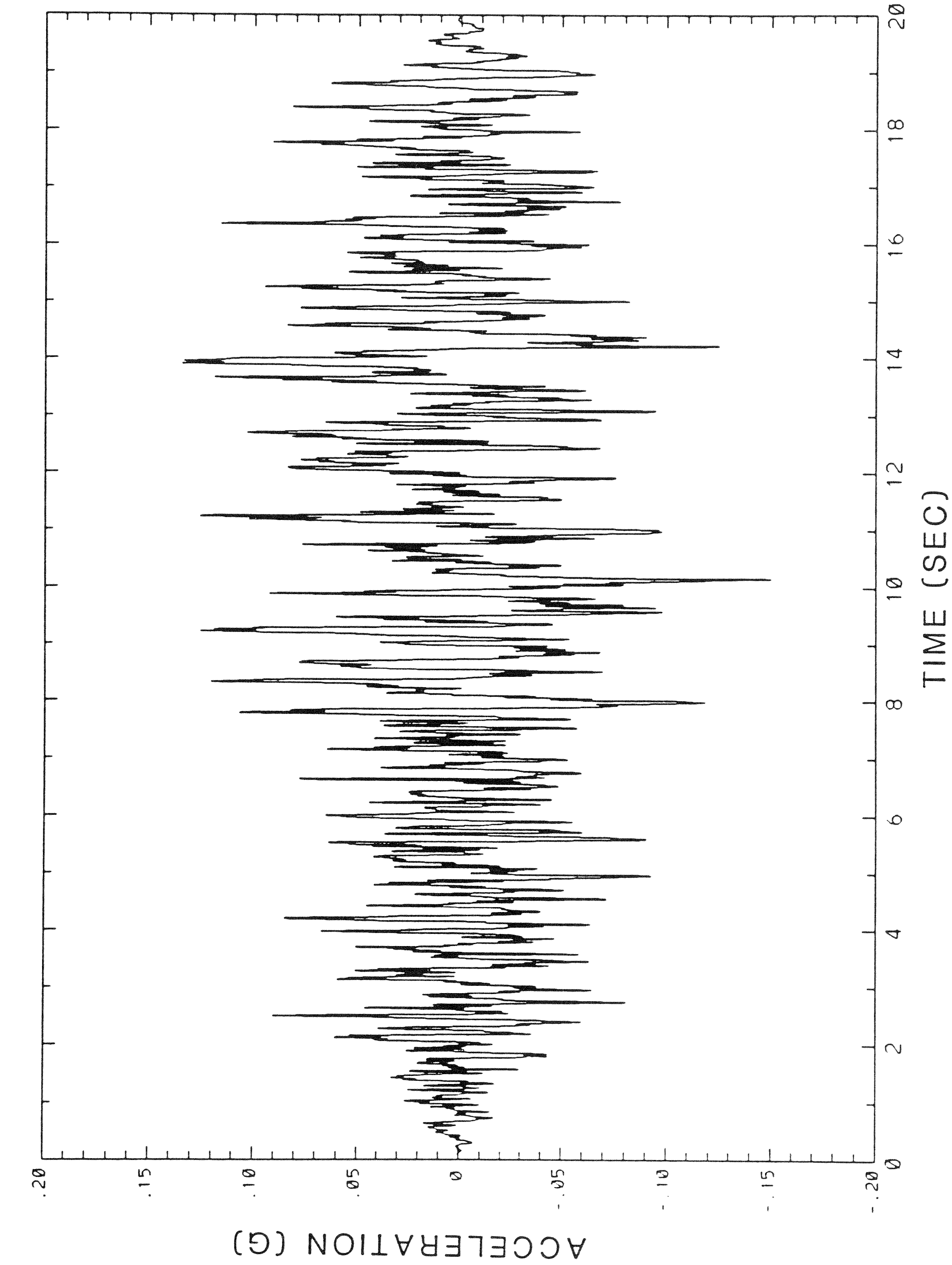

FIGURE 4-2 A Sample of Synthetic Earthquakes

SECTION 5
DETERMINATION OF EMPIRICAL FORMULA

In this study, the response modification factor R is defined as

$$R = \frac{V_l}{V_n} \tag{5.1}$$

where V_n is the absolute maximum base shear obtained from a nonlinear time history analysis, while V_l is the corresponding value obtained from a linear time history analysis using the same earthquake accelerogram. The response modification factor R is influenced by many parameters describing the earthquake-structure system. In this study, R is considered as a function of the following parameters:

$$R = f(\mu_m, \zeta, T) \tag{5.2}$$

where ζ is the viscous damping ratio and μ_m is the maximum story ductility ratio, which is the largest value of all story ductility ratios. T is the earthquake-structure period ratio defined as

$$T = \frac{T_s}{T_g} \tag{5.3}$$

in which

$T_s =$ fundamental period of structure

$T_g =$ dominant period of earthquake motion, $T_g = 2\pi / \omega_g$

5.1 Generation of Response Data

In this study, twelve structural models with various fundamental periods and viscous damping ratios are used to represent typical low-rise to mid high-rise reinforced concrete structures. On the other hand, 90 synthetic earthquake motions are generated from three power spectra, which have different dominant periods due to soil conditions. From the nonlinear time history analysis of a structural model subject to a synthetic earthquake, the absolute maximum base shear V_n and the maximum ductility ratio μ_m are obtained, while V_l is obtained from the corresponding linear analysis. Then, the response modification factor R can be determined by using Eq. (5.1). The nonlinear and corresponding linear time history analyses are carried out for all 12 structural models under 90 earthquakes. Thus, a total of 1080 runs has been performed and results are shown in Appendix B. However, there are 20 runs in which structures remain in the elastic range; therefore these 20 runs are excluded from data base for the regression analysis.

5.2 Multivariate Nonlinear Regression Analysis

For regression analysis the following form is assumed

$$\ell n R = [e^{-\theta_1 T} - e^{-\theta_2 T} - \theta_3 \zeta] \ell n \mu_m \tag{5.4}$$

where θ_1, θ_2, and θ_3 are unknown coefficients to be determined from multivariate nonlinear regression analysis [16]. The curve obtained from the regression analysis represents the mean curve on the basis of available data. Dispersion of data about the regression curve is measured by the conditional variance. From the scattergram of response data as shown in figure 5-1, it is observed that the data are more scattered with the increasing values of $\ell n \mu_m$. Thus, the conditional variance of $\ell n R$ is not constant and is assumed as function of $\ell n \mu_m$.

$$Var(\ell n R | \ell n \mu_m) = s(\ell n \mu_m)^2 \tag{5.5}$$

where s is an unknown coefficient. Using the subroutine DRNLIN in the International Mathematical and Statistical Libraries (IMSL) [17], which implements the modified Levenberg-Marquardt algorithm, the unknown regression coefficients in Eqs. (5.4) and (5.5) are determined as follows:

$$\begin{aligned} \theta_1 &= 0.1857 \\ \theta_2 &= 2.1673 \\ \theta_3 &= 0.0276 \\ s &= 0.0128 \end{aligned} \tag{5.6}$$

Thus, the empirical formula for the response modification factor R is

$$\ell n R = [e^{-0.1857 T} - e^{-2.1673 T} - 0.0276 \zeta] \ell n \mu_m \tag{5.7}$$

$$Var(\ell n R | \ell n \mu_m) = 0.0128(\ell n \mu_m)^2 \tag{5.8}$$

From Eq. (5.8), the conditional standard deviation $\sigma_{lnR|ln\mu_m}$ is equal to 0.113 $ln \mu_m$.

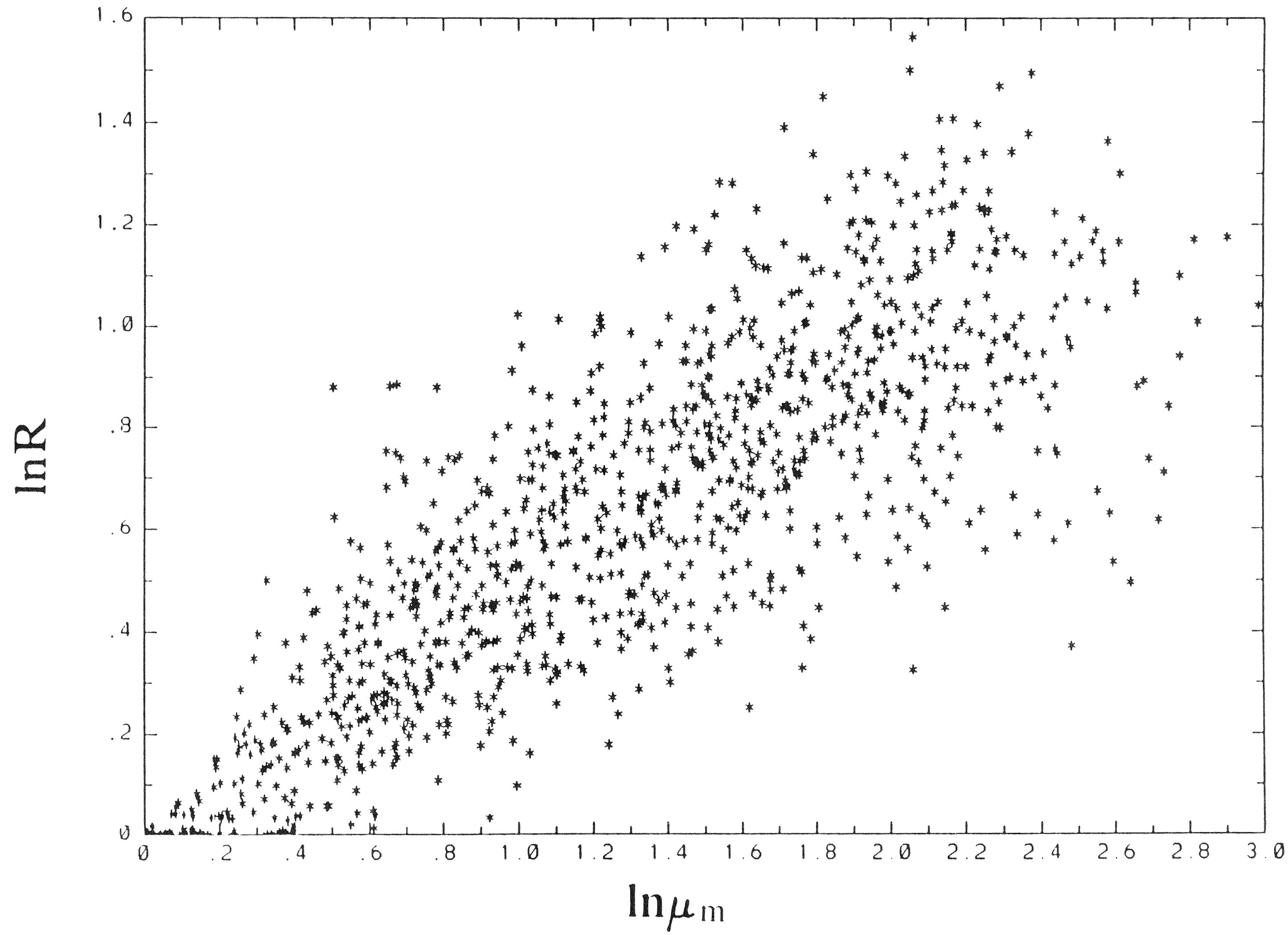

FIGURE 5-1 Scattergram of Structural Response Data

SECTION 6
ILLUSTRATION AND COMPARISON

In this section, a four-story structure (structural model no. 5) and a ten-story structure (structural model no. 11) are utilized to demonstrate the proposed formula for the R factor applicable to reinforced concrete structures. In addition, the proposed formula is compared with Newmark-Hall formula [5] and Newmark-Riddell formula [3,4]. These three formulas are briefly described below.

(1) Proposed Formula

The formula for the R factor proposed in this study, i.e., Eqs. (5.7) and (5.8), is a function of the maximum ductility ratio, the viscous damping ratio, the fundamental period of structure and the dominant frequency of earthquake motion. Furthermore, variation of R factor is expressed as a function of the maximum ductility ratio.

(2) Newmark-Hall Formula

For the purpose of deriving an inelastic response spectrum, Newmark and Hall investigated the elasto-plastic response of SDF systems and suggested the following response modification factor. In the displacement and velocity regions, the maximum displacement of an elasto-plastic system is assumed to be the same as the maximum displacement of an elastic system; thus the response modification factor R is

$$R = \mu \tag{6.1}$$

where μ is the ductility ratio of the SDF system. In the acceleration region, the strain energy accumulated in an elasto-plastic system is assumed to be equivalent to the strain energy of an elastic system and the R factor is expressed as

$$R = (2\mu - 1)^{\frac{1}{2}} \tag{6.2}$$

(3) Newmark-Riddell Formula

Newmark and Riddell conducted a study to improve Newmark-Hall formula. From a statistical analysis of the response data obtained from SDF systems with various hysteretic models and subject to actual earthquake records, Newmark and Riddell suggested an empirical formula in terms of the ductility ratio μ and the viscous damping ratio ζ of the structure.

In the acceleration region, Newmark-Riddell formula is

$$R = [(q+1)\mu - q]^r \tag{6.3}$$

and the coefficients q and r were determined as

$$q = 3.00\zeta^{-0.3}; \quad r = 0.48\zeta^{-0.08} \tag{6.4}$$

In the velocity region, Newmark-Riddell formula has the same form as Eq. (6.3) with the following expressions for coefficients q and r.

$$q = 2.70\zeta^{-0.4}; \quad r = 0.66\zeta^{-0.04} \tag{6.5}$$

In the displacement region, Newmark-Riddell formula is given as

$$R = p\mu^r \tag{6.6}$$

and the coefficients p and r were determined as

$$p = 1.15\zeta^{-0.055}; \quad r = 1.07 \tag{6.7}$$

6.1 Four-Story Structure

The first structure utilized for demonstration and comparison is a four-story structure with properties the same as structural model no. 5. The fundamental period of the structure is 0.6 second and the viscous damping ratio is five percent. The synthetic earthquakes are generated from the K-T spectrum with $\omega_g = 5\pi$ and $\zeta_g = 0.6$. This spectrum represents a deep cohesionless soil condition. The duration of strong motion is taken as 15 seconds and the total duration of earthquake accelerogram is 20 seconds. Seven levels of peak ground acceleration (PGA) are used: 0.075, 0.1, 0.125, 0.15, 0175, 0.2 and 0.225 g. For each PGA level, 25 synthetic earthquakes are generated. Thus, a total of 175 earthquakes is produced. The nonlinear and corresponding linear analyses of the four-story structure subject to each synthetic earthquake are performed. Therefore, 175 maximum ductility ratios μ_m and the response modification factors R are obtained and plotted in figure 6-1. Three empirical formulas, i.e. the proposed, Newmark-Hall and Newmark-Riddell formulas, are also plotted in this figure. It is noted that the fundamental period of the structure is 0.6 second; thus the expression applicable in the velocity region in Newmark-Hall formula and Newmark-Riddell formula are used. From figure 6-1, it can be seen that the proposed formula fits the data reasonably well; while Newmark-Hall and Newmark-Riddell formulas are on the upper side of the data. It means that for a given ductility ratio, the response modification factors predicted by Newmark-Hall formula or Newmark-Riddell formula are larger than the actual value. Thus, the response modification factor predicted from these two formulas are unconservative.

6.2 Ten-Story Structure

The second structure used for comparison is a ten-story structure (structural model no. 11). The fundamental period of this structure is 1.2 second and the viscous damping ratio is five percent. The K-T power spectrum representing soft soil condition is used; thus ω_g is taken as 2.4π rad/sec and ζ_g is set equal to 0.85. The duration of strong motion is 20 seconds and the total duration of earthquake is 25 seconds. Similar to previous case, 175 earthquakes are generated for seven PGA levels: 0.05, 0.075, 0.10, 0.125, 0.15, 0.175 and 0.2 g. The response data are obtained from both nonlinear and corresponding linear time history analyses. For PGA equal to 0.05 g, 19 cases remain in the elastic range, and thus these cases are excluded from the data. Figure 6-2 shows the plot of the remaining 156 cases of μ_m and R. Three empirical formulas are also plotted in the figure. The results are similar to that obtained from the four-story structure. That is, the proposed formula fits the data reasonably well; while Newmark-Hall formula and Newmark-Riddell formula give unconservative prediction.

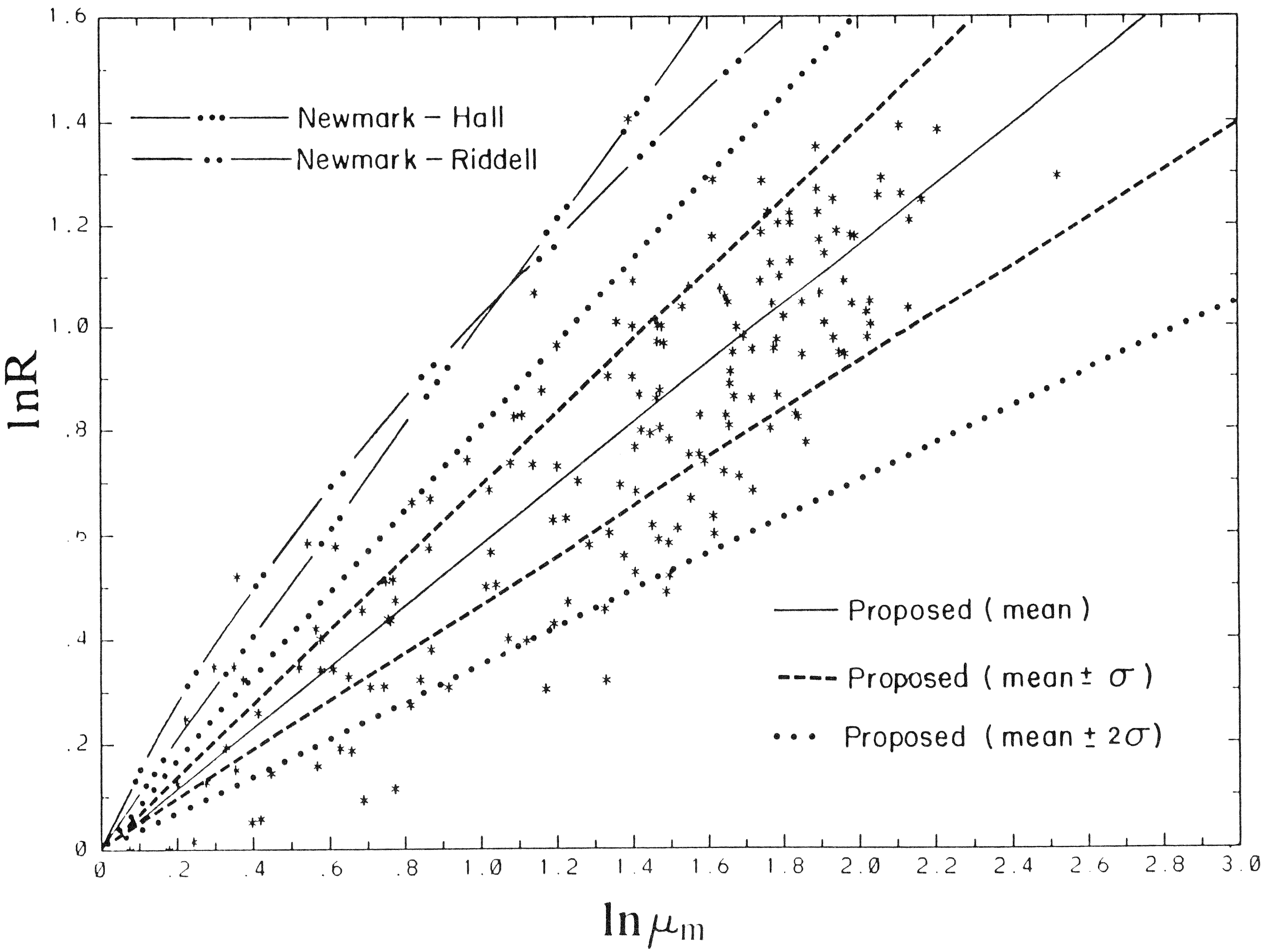

FIGURE 6-1 Comparison of Formulas for R Factor (4-Story Structure)

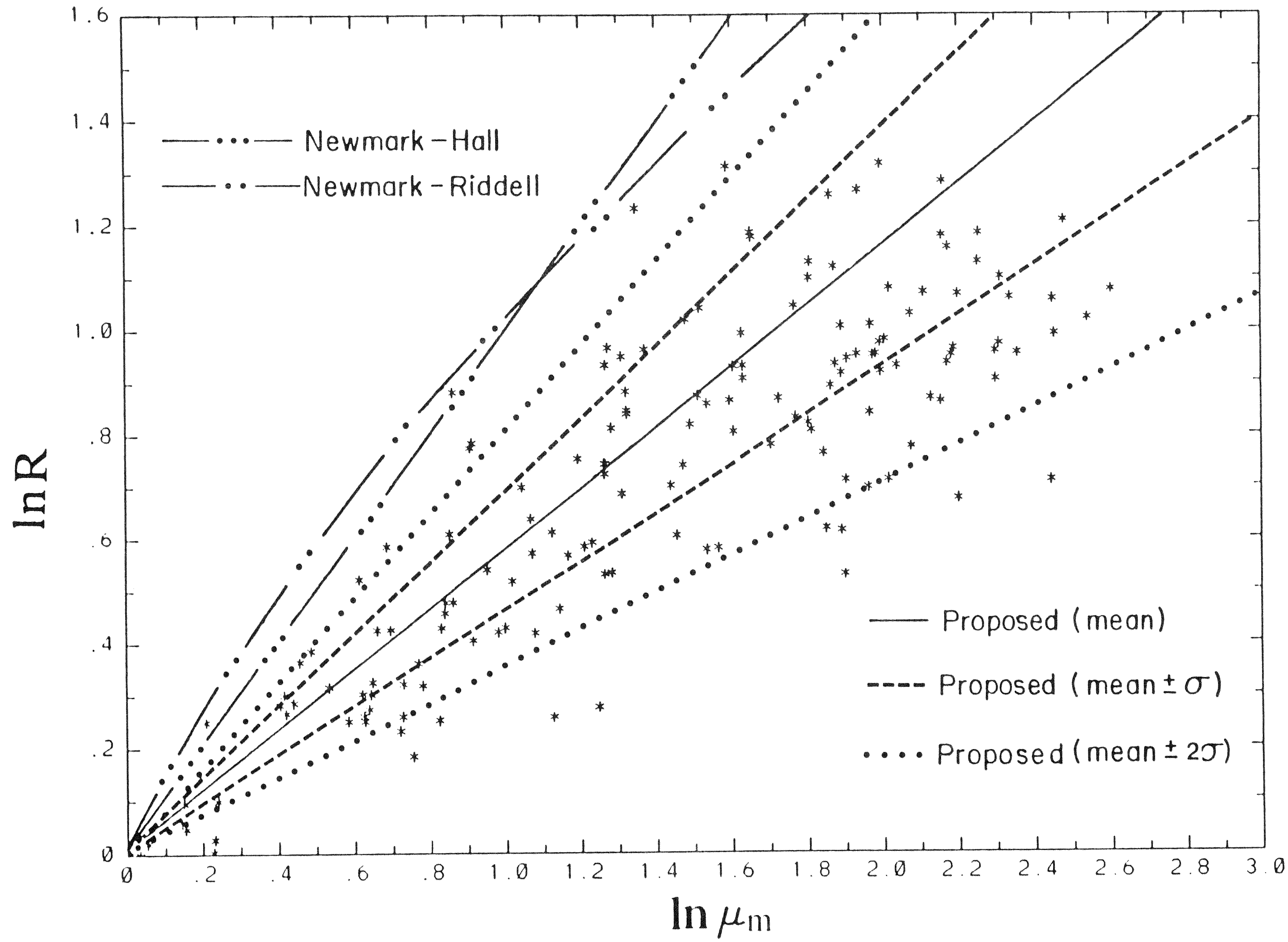

FIGURE 6-2 Comparison of Formulas for R Factor (10-Story Structure)

SECTION 7
RECOMMENDATION FOR EARTHQUAKE RESISTANT DESIGN

The seismic design criteria specified in building codes such as the NEHRP Recommended Provisions utilize the response modification factor to include the effect of nonlinear deformation into the design procedure. Table 7-I shows the response modification factors specified in the NEHRP Recommended Provisions for reinforced concrete structures. It is noted that a constant value is assigned to each type of structural system and seismic resisting system. This constant R factor reflects an unknown and unspecified ductility ratio that the structure is allowed to reach under the design earthquake load.

In this study, the response modification factor determined from Eq. (5.7) is recommended for use in the earthquake-resistant design of buildings. As shown in Eq. (5.7), the recommended R factor is a function of the maximum ductility ratio μ_m, the viscous damping ratio ζ, and the earthquake-structure period ratio T. For the viscous damping ratio of five percent, the response modification factors are plotted as a function of T for various levels of the maximum ductility ratio as shown in figure 7-1.

It is noted that when T is small, e.g., less than 0.5, the R factor is also small. Structures with small T represent very stiff structures. The response of this type of structure to earthquakes is in rigid mode. The nonlinear effect is not significant and the nonlinear response is close to the linear response. Thus, the response modification factor R is close to 1.0. On the other hand, structures with large T represent very flexible structures such as high-rise buildings. This type of structure subject to earthquakes will produce larger deformation and less force as compared with a stiff structure with similar geometry. Thus, the R factor also tends to be smaller. The R factor varies with T significantly, especially in the case of large ductility ratios. Therefore, the earthquake-structure period ratio T is an important parameter to be considered in determining the R value.

Figure 7-1 is a useful tool for seismic design. For example, if a building frame system with reinforced concrete shear wall is allowed to have the maximum ductility ratio of 4, which may be correspondent to moderate structural damage; then the R factor displayed by the curve with $\mu = 4$ in figure 7-1 can be utilized to determine the design base shear.

It is noted that the response modification factor recommended for design represents the mean value. Variability of the R factor expressed in Eq. (5.8) is not included. This

TABLE 7-I NEHRP Response Modification Factors

Type of Structural System	Seismic Resisting System	R
Bearing Wall System	Reinforced Concrete Shear Walls	4-1/2
Building Frame System	Reinforced Concrete Shear Walls	5-1/2
Moment Resisting Frame	Special Moment Frames of Reinforced Concrete	8
	Ordinary Moment Frames of Reinforced Concrete	2
	Intermediate Moment Frames of Reinforced Concrete	4
Dual System	A Special Moment Frame and Reinforced Concrete Shear Walls	8
	Intermediate Moment Frame and Reinforced Concrete Shear Walls	6

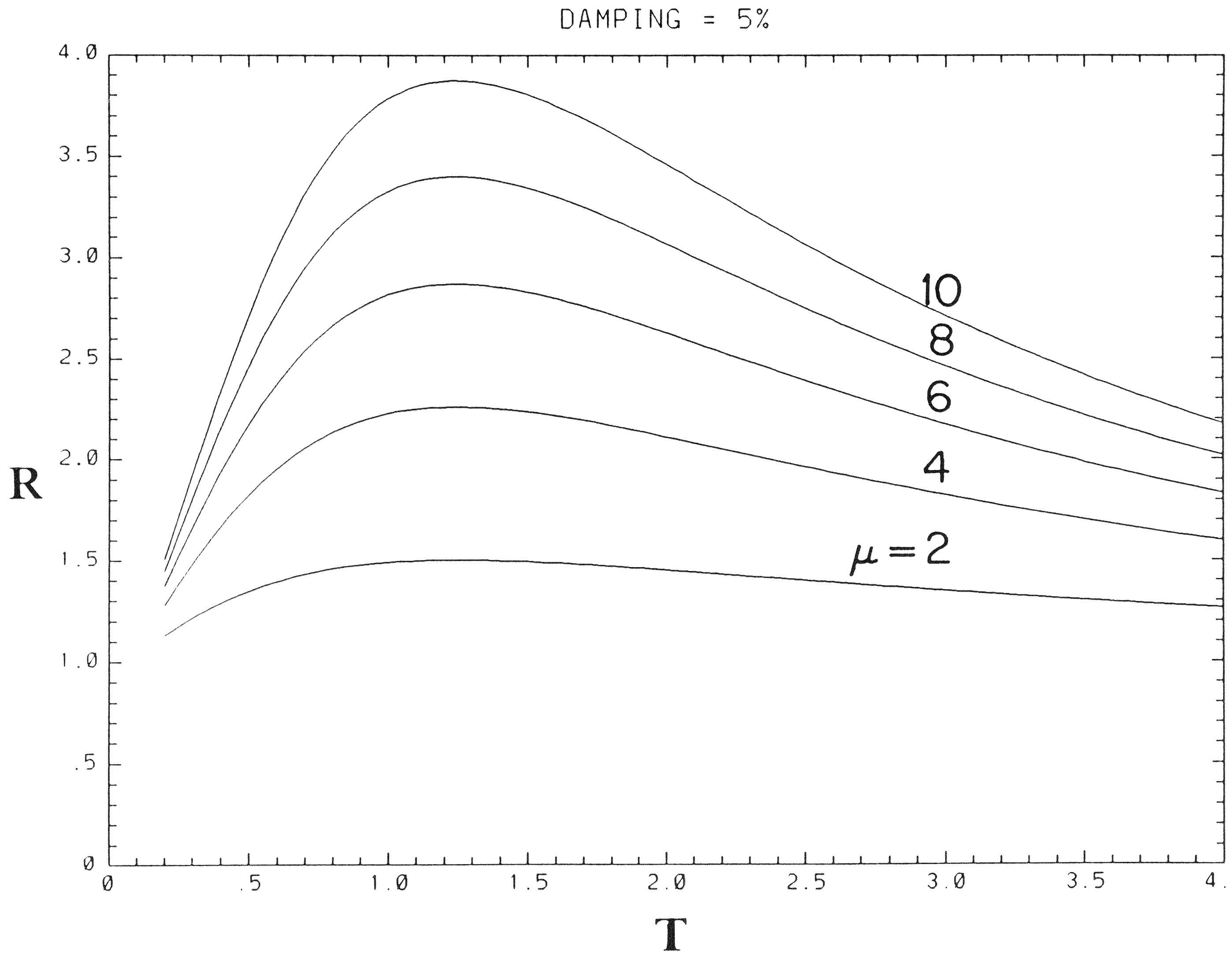

FIGURE 7-1 Recommended R Factors for Design

variability should also be taken into consideration in the code development. For example, if the load resistance factor design (LRFD) format is used in seismic design criteria; then variability of R factors can be included in the seismic load factor.

As shown in table 7-I, most of the R factors specified in the NEHRP Recommended Provisions are larger than 4, while the R factor in figure 7-1 is less than 4 even though the maximum ductility ratio of 10 is allowed. Thus, the response modification factor specified in the NEHRP Recommended Provisions seems to be too large and unconservative. It should be noted that the use of large R factor in the design of buildings does not imply the building is unsafe since there are other safety factors built in the seismic design criteria. Nevertheless, the specification of more reasonable R factors in the seismic design provisions is warranted.

SECTION 8
CONCLUSIONS

This report presents a statistical evaluation of the response modification factor R for reinforced concrete structures. Twelve structural models with various dynamic characteristics are first constructed. Next, 90 synthetic earthquakes are generated from three power spectra representing different soil conditions. Then, the nonlinear and corresponding linear time histories analyses are performed to produce structural response data. On the basis of these data, an empirical formula for the response modification factor is established from the multivariate nonlinear regression analysis. The empirical formula describing the mean value of the R factor is a function of the maximum ductility ratio, the viscous damping ratio and the earthquake-structure period ratio. In addition, variation of the R factor in terms of the maximum ductility ratio is also established from the multivariate nonlinear regression analysis. The response modification factor is used in the seismic design criteria to include the effect of nonlinear deformation into the design. The response modification factors estimated from the empirical formula can be used to improve the seismic design criteria such as the NEHRP Recommended Provisions. The authors believe that most of the R factors specified in the current NEHRP provisions are too large and unconservative. Thus, the specification of more reasonable R factors in the seismic design provisions is warranted.

In this study, the response modification factors are established for reinforced concrete structures. The response modification factors applicable to other structures such as steel structures can be established following the same approach. In addition, the actual nonlinear deformation of a structure under design earthquake will be larger than that determined from the equivalent linear analysis. In order to evaluate the actual nonlinear deformation, the NEHRP Recommended Provisions utilizes the deflection amplification factor C_d to modify the deflection evaluated from equivalent linear analysis. In order to improve the seismic design criteria for buildings, the deflection amplification factor C_d needs to be carefully evaluated.

SECTION 9
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APPENDIX A

STORY YIELDING STRENGTH

The structural models used in this study are designed according to the seismic provisions of ANSI A58.1 standard. The design seismic base shear V is:

$$V = ZIKCSW \tag{A.1}$$

where

- V: total shear force at the base
- Z: zone factor
- I: importance factor
- K: building system factor
- C: numerical coefficient
- S: soil factor
- W: total dead load of the building

The zone factor Z is assumed to be 0.5, which is one-half of that used in seismic zone 4 ($Z = 1.0$). The corresponding effective peak acceleration (EPA) is equal to 0.2g. The importance factor I and building system factor K are taken to be 1.0. The soil condition at the site is assumed to be classified as S_2. Thus the soil factor S is equal to 1.2. The coefficient C is determined by

$$C = \frac{1}{15\sqrt{T_s}} \leq 0.12 \tag{A.2}$$

in which T_s is the fundamental period of the structure as established in table 3-I for structures selected in this study. Furthermore, ANSI A58.1 specifies that the product CS needs not exceed 0.14. The seismic base shear coefficients, i.e., $ZIKCS$, for 12 structures are determined and given in table A-I.

The base shear is distributed over the height of the structure and the lateral force acting at the j-th floor is

$$F_j = \frac{(V - F_t)W_j h_j}{\sum_{i=1}^{n} W_i h_i} \tag{A.3}$$

where

- F_j: lateral force applied at level j
- F_t: additional concentrated lateral force at the top of structure

TABLE A-I Seismic Base Shear Coefficient

Structure	Fundamental Period T_s (sec.)	Base Shear Coefficient
1-3	0.3	0.07
4-6	0.6	0.0516
7-9	0.9	0.0422
10-12	1.2	0.0365

APPENDIX B

STRUCTURAL RESPONSE DATA

Structure 1 (T_s = 0.3 sec, ζ = 3%); Earthquakes (T_g = 0.25 sec)

EQ	μ_m	R	EQ	μ_m	R
1	2.513	1.953	16	5.077	3.105
2	1.977	2.095	17	4.442	2.380
3	2.239	1.222	18	2.946	2.366
4	2.319	2.105	19	7.130	2.892
5	2.569	1.390	20	6.738	2.757
6	2.804	1.648	21	10.745	4.458
7	2.288	2.085	22	6.703	2.475
8	1.383	1.650	23	4.518	3.193
9	3.388	1.823	24	6.979	2.979
10	2.814	2.007	25	9.470	3.820
11	5.225	1.956	26	4.666	2.221
12	4.882	2.873	27	6.126	3.042
13	2.816	2.397	28	4.844	2.925
14	4.063	2.769	29	6.672	3.344
15	5.010	2.374	30	4.593	3.385

Structure 1 (T_s = 0.3 sec, ζ = 3%); Earthquakes (T_g = 0.4 sec)

EQ	μ_m	R	EQ	μ_m	R
31	2.980	1.912	46	5.643	2.295
32	2.789	2.005	47	5.822	2.474
33	3.640	1.472	48	5.701	2.695
34	2.351	1.560	49	6.475	2.533
35	2.072	1.569	50	4.526	2.195
36	2.266	1.666	51	8.462	3.843
37	1.966	1.678	52	8.107	2.444
38	2.895	1.779	53	8.731	4.085
39	3.161	1.977	54	10.058	3.242
40	2.090	1.830	55	7.916	3.158
41	6.650	2.728	56	11.983	3.074
42	4.909	2.689	57	12.348	3.360
43	4.339	2.705	58	12.798	3.275
44	9.864	4.350	59	9.744	2.639
45	7.117	3.226	60	9.054	3.772

Structure 1 ($T_s = 0.3$ sec, $\zeta = 3\%$); Earthquakes ($T_g = 0.83$ sec)

EQ	μ_m	R	EQ	μ_m	R
61	2.090	1.284	76	4.069	1.788
62	2.323	1.709	77	7.890	2.143
63	1.783	1.257	78	6.531	2.608
64	2.093	1.456	79	6.426	1.863
65	1.713	1.434	80	7.727	1.754
66	2.699	1.695	81	8.935	2.321
67	1.353	1.486	82	11.504	2.110
68	3.749	1.333	83	15.580	2.320
69	2.060	1.641	84	9.386	1.889
70	1.757	1.593	85	10.917	1.875
71	3.871	1.810	86	15.373	2.036
72	6.986	2.377	87	11.831	2.655
73	6.738	2.231	88	8.794	2.403
74	6.321	2.232	89	9.182	2.320
75	9.102	1.842	90	7.619	2.396

Structure 2 (T_s = 0.3 sec, ζ = 5%); Earthquakes (T_g = 0.25 sec)

EQ	μ_m	R	EQ	μ_m	R
1	1.457	1.460	16	3.299	2.479
2	1.924	1.540	17	3.803	2.226
3	1.554	1.059	18	2.122	2.082
4	2.744	1.597	19	5.772	2.181
5	1.362	1.196	20	6.348	2.492
6	1.897	1.256	21	6.618	3.324
7	2.285	1.754	22	4.989	2.222
8	1.342	1.207	23	4.251	2.618
9	1.645	1.422	24	4.426	2.536
10	1.564	1.549	25	6.641	2.850
11	3.314	1.526	26	3.578	1.960
12	4.314	2.421	27	5.627	2.474
13	2.947	1.883	28	5.177	2.398
14	3.376	2.512	29	5.886	2.741
15	3.991	1.983	30	4.385	2.527

Structure 2 (T_s = 0.3 sec, ζ = 5%); Earthquakes (T_g = 0.4 sec)

EQ	μ_m	R	EQ	μ_m	R
31	2.524	1.575	46	5.063	2.077
32	2.660	1.687	47	5.207	1.944
33	2.045	1.406	48	4.850	2.266
34	1.858	1.318	49	5.090	2.071
35	2.065	1.323	50	4.024	2.050
36	1.891	1.299	51	5.146	3.425
37	1.852	1.542	52	7.253	2.325
38	2.288	1.404	53	6.230	3.494
39	1.711	1.571	54	10.616	2.571
40	1.580	1.557	55	6.975	2.376
41	4.261	2.251	56	10.045	2.444
42	4.377	2.205	57	7.894	2.828
43	4.198	2.201	58	8.693	3.259
44	8.472	3.416	59	8.714	2.188
45	4.757	2.631	60	9.742	3.149

Structure 2 (T_s = 0.3 sec, ζ = 5%); Earthquakes (T_g = 0.83 sec)

EQ	μ_m	R	EQ	μ_m	R
61	1.269	1.108	76	2.815	1.512
62	1.673	1.398	77	6.532	1.790
63	1.506	1.184	78	5.525	2.050
64	1.675	1.165	79	5.951	1.471
65	1.286	1.192	80	5.343	1.620
66	1.729	1.241	81	7.420	1.889
67	1.312	1.197	82	8.555	1.921
68	2.795	1.176	83	12.835	1.962
69	1.958	1.341	84	8.533	1.565
70	1.552	1.248	85	9.492	1.749
71	4.035	1.604	86	14.036	1.643
72	5.779	2.028	87	10.901	2.122
73	5.562	2.002	88	8.139	1.837
74	5.276	1.870	89	8.026	1.864
75	7.486	1.628	90	6.781	2.128

Structure 3 (T_s = 0.3 sec, ζ = 7%); Earthquakes (T_g = 0.25 sec)

EQ	μ_m	R	EQ	μ_m	R
1	1.303	1.224	16	2.252	2.098
2	1.272	1.212	17	3.203	2.080
3	1.250	1.031	18	1.731	1.779
4	1.482	1.363	19	4.556	1.771
5	1.136	1.038	20	3.505	2.106
6	1.216	1.039	21	3.670	2.686
7	2.030	1.565	22	3.222	2.006
8	1.042	1.000	23	3.815	2.245
9	1.494	1.175	24	3.770	2.360
10	1.375	1.268	25	3.992	2.241
11	1.766	1.252	26	3.385	1.749
12	4.444	2.076	27	4.857	2.298
13	2.470	1.559	28	5.387	2.079
14	2.927	2.156	29	4.874	2.355
15	2.561	1.662	30	4.703	2.147

Structure 3 (T_s = 0.3 sec, ζ = 7%); Earthquakes (T_g = 0.4 sec)

EQ	μ_m	R	EQ	μ_m	R
31	2.023	1.345	46	4.910	1.867
32	2.444	1.489	47	4.299	1.702
33	1.963	1.296	48	3.977	1.971
34	1.325	1.174	49	4.387	1.860
35	1.710	1.214	50	3.583	1.905
36	1.202	1.101	51	5.249	3.051
37	1.616	1.406	52	6.767	2.287
38	2.047	1.242	53	6.384	3.011
39	1.515	1.356	54	9.877	2.224
40	1.291	1.332	55	5.596	2.474
41	3.663	2.336	56	8.839	2.101
42	3.742	1.894	57	5.973	2.536
43	3.922	1.916	58	7.365	2.981
44	4.557	2.814	59	7.807	2.099
45	2.546	2.190	60	8.937	2.746

Structure 3 (T_s = 0.3 sec, ζ = 7%); Earthquakes (T_g = 0.83 sec)

EQ	μ_m	R	EQ	μ_m	R
61	1.019	1.017	76	1.770	1.348
62	1.603	1.211	77	6.081	1.564
63	1.296	1.119	78	3.246	1.791
64	1.072	1.043	79	5.044	1.286
65	1.155	1.070	80	4.510	1.503
66	1.093	1.067	81	7.339	1.709
67	1.065	1.001	82	7.520	1.794
68	1.952	1.200	83	11.835	1.841
69	1.432	1.165	84	7.836	1.384
70	1.148	1.086	85	8.134	1.693
71	3.033	1.466	86	11.939	1.450
72	4.760	1.825	87	6.914	1.873
73	4.563	1.807	88	6.738	1.727
74	5.028	1.704	89	7.752	1.896
75	5.836	1.508	90	5.448	1.969

Structure 4 (T_s = 0.6 sec, ζ = 3%); Earthquakes (T_g = 0.25 sec)

EQ	μ_m	R	EQ	μ_m	R
1	2.239	1.463	16	4.625	1.559
2	2.379	1.564	17	6.574	2.372
3	1.833	1.153	18	1.652	2.410
4	2.915	1.422	19	2.784	2.119
5	2.191	1.116	20	2.984	1.864
6	1.434	1.004	21	6.722	3.254
7	2.514	1.034	22	5.772	1.685
8	1.958	1.167	23	6.005	2.576
9	2.520	1.696	24	5.786	2.081
10	4.417	1.908	25	5.505	2.647
11	2.897	1.752	26	4.473	2.372
12	4.170	2.146	27	5.831	2.353
13	2.316	1.853	28	3.329	2.683
14	5.629	1.822	29	5.998	2.202
15	2.866	1.854	30	4.828	1.681

Structure 4 (T_s = 0.6 sec, ζ = 3%); Earthquakes (T_g = 0.4 sec)

EQ	μ_m	R	EQ	μ_m	R
31	4.444	2.059	46	8.711	3.449
32	4.525	2.123	47	4.437	2.418
33	2.558	1.578	48	7.031	3.175
34	4.662	1.854	49	4.653	3.608
35	2.500	1.973	50	4.778	1.998
36	4.152	1.994	51	9.302	4.040
37	4.221	2.539	52	9.812	3.225
38	3.231	1.381	53	5.811	3.110
39	2.623	1.644	54	7.824	4.779
40	1.901	2.123	55	7.667	3.798
41	5.771	2.913	56	8.265	3.550
42	5.648	2.529	57	9.517	3.426
43	2.668	2.493	58	8.790	3.454
44	6.231	2.454	59	6.149	4.266
45	4.485	3.158	60	7.771	4.485

Structure 4 ($T_s = 0.6$ sec, $\zeta = 3\%$); Earthquakes ($T_g = 0.83$ sec)

EQ	μ_m	R	EQ	μ_m	R
61	2.434	2.004	76	5.648	2.594
62	5.579	1.979	77	9.497	3.399
63	2.997	2.107	78	7.440	3.315
64	5.571	2.321	79	7.961	3.033
65	3.055	1.939	80	9.619	3.045
66	3.321	2.258	81	9.376	3.164
67	4.511	2.461	82	8.500	3.608
68	5.684	2.046	83	8.421	4.083
69	2.536	2.090	84	9.613	3.415
70	3.184	2.185	85	9.602	3.547
71	5.203	2.619	86	11.451	3.402
72	6.824	2.950	87	10.660	3.965
73	7.266	2.835	88	8.264	3.104
74	6.721	3.144	89	8.713	3.210
75	10.052	2.670	90	9.799	3.144

Structure 5 (T_s = 0.6 sec, ζ = 5%); Earthquakes (T_g = 0.25 sec)

EQ	μ_m	R	EQ	μ_m	R
1	1.701	1.163	16	3.906	1.447
2	1.646	1.272	17	5.250	2.157
3	1.413	1.034	18	2.100	1.706
4	2.025	1.181	19	1.658	1.865
5	1.265	1.033	20	2.726	1.470
6	1.132	1.008	21	5.866	2.444
7	1.840	1.012	22	5.220	1.578
8	1.483	1.012	23	4.979	2.153
9	1.849	1.287	24	4.889	1.917
10	3.039	1.480	25	5.047	2.123
11	2.529	1.251	26	3.829	1.949
12	2.887	1.890	27	4.697	1.750
13	1.528	1.476	28	3.159	2.047
14	3.877	1.503	29	4.645	2.003
15	2.461	1.453	30	4.634	1.463

Structure 5 (T_s = 0.6 sec, ζ = 5%); Earthquakes (T_g = 0.4 sec)

EQ	μ_m	R	EQ	μ_m	R
31	3.712	1.796	46	7.074	2.546
32	4.053	1.699	47	3.242	1.959
33	1.410	1.203	48	5.376	2.467
34	3.572	1.489	49	3.380	2.770
35	2.188	1.629	50	3.840	1.667
36	2.761	1.504	51	6.877	3.091
37	4.873	2.053	52	8.381	2.851
38	3.455	1.196	53	5.158	2.437
39	1.932	1.304	54	6.626	3.661
40	2.076	1.633	55	7.189	3.092
41	4.754	2.364	56	5.946	2.835
42	4.332	2.306	57	7.607	2.736
43	3.295	2.142	58	7.883	2.694
44	5.698	2.084	59	4.815	3.601
45	1.956	2.423	60	7.874	3.316

Structure 5 (T_s = 0.6 sec, ζ = 5%); Earthquakes (T_g = 0.83 sec)

EQ	μ_m	R	EQ	μ_m	R
61	2.209	1.774	76	4.101	2.134
62	2.958	1.599	77	7.356	2.693
63	2.195	1.697	78	7.197	2.603
64	2.420	1.753	79	5.509	2.846
65	2.500	1.742	80	6.748	2.577
66	2.687	1.816	81	8.546	2.508
67	3.358	1.857	82	7.561	2.408
68	4.310	1.576	83	7.333	3.654
69	2.426	1.605	84	7.441	2.625
70	2.658	1.771	85	7.233	2.674
71	4.616	2.276	86	7.891	3.072
72	6.765	2.599	87	9.249	3.066
73	5.519	2.313	88	7.257	2.428
74	6.027	2.529	89	9.112	2.697
75	8.079	2.252	90	8.103	2.515

Structure 6 (T_s = 0.6 sec, ζ = 7%); Earthquakes (T_g = 0.25 sec)

EQ	μ_m	R	EQ	μ_m	R
1	1.246	1.047	16	3.007	1.371
2	1.086	1.043	17	4.143	1.967
3	1.159	1.004	18	1.511	1.392
4	1.731	1.020	19	1.541	1.617
5	1.010	1.000	20	2.446	1.292
6	0.991	*	21	4.789	2.008
7	1.444	1.008	22	4.281	1.427
8	1.052	1.000	23	3.962	1.858
9	1.336	1.047	24	4.148	1.766
10	2.126	1.213	25	3.904	1.799
11	1.760	1.044	26	3.474	1.735
12	2.890	1.633	27	3.673	1.549
13	1.320	1.245	28	3.768	1.784
14	2.124	1.330	29	3.986	1.817
15	2.252	1.245	30	4.330	1.435

Note: * indicates structure remains elastic

Structure 6 (T_s = 0.6 sec, ζ = 7%); Earthquakes (T_g = 0.4 sec)

EQ	μ_m	R	EQ	μ_m	R
31	3.216	1.590	46	5.847	2.123
32	1.961	1.430	47	2.559	1.699
33	1.138	1.010	48	3.868	2.085
34	1.674	1.261	49	2.642	2.230
35	1.912	1.418	50	2.817	1.485
36	1.786	1.354	51	5.437	2.562
37	2.915	1.762	52	6.957	2.538
38	2.702	1.103	53	3.646	2.199
39	1.465	1.233	54	5.305	3.046
40	1.725	1.356	55	6.590	2.662
41	4.003	1.974	56	5.101	2.661
42	2.945	2.114	57	6.199	2.445
43	3.053	1.887	58	9.722	2.418
44	4.232	1.782	59	4.023	3.177
45	1.992	2.017	60	7.502	2.816

Structure 6 (T_s = 0.6 sec, ζ = 7%); Earthquakes (T_g = 0.83 sec)

EQ	μ_m	R	EQ	μ_m	R
61	2.050	1.620	76	3.747	1.942
62	1.681	1.388	77	5.594	2.322
63	1.338	1.416	78	6.185	2.226
64	2.260	1.535	79	4.520	2.574
65	1.792	1.573	80	4.561	2.095
66	1.996	1.639	81	7.089	2.194
67	2.067	1.537	82	6.305	2.408
68	2.922	1.396	83	6.905	3.350
69	1.929	1.353	84	6.828	2.315
70	2.183	1.602	85	6.319	2.268
71	4.563	1.961	86	5.853	2.734
72	5.925	2.335	87	8.242	2.598
73	5.037	2.016	88	6.697	2.020
74	5.433	2.196	89	8.823	2.510
75	5.088	1.970	90	6.279	2.164

Structure 7 (T_s = 0.9 sec, ζ = 3%); Earthquakes (T_g = 0.4 sec)

EQ	μ_m	R	EQ	μ_m	R
31	1.711	1.529	46	14.571	2.437
32	3.300	1.579	47	5.088	2.361
33	1.629	1.450	48	2.948	2.239
34	2.198	1.463	49	4.771	2.189
35	3.780	1.634	50	3.053	2.024
36	2.064	1.586	51	19.832	2.832
37	3.566	1.545	52	11.019	2.366
38	2.779	1.380	53	18.229	3.239
39	2.571	1.670	54	11.777	2.873
40	4.734	1.599	55	11.432	2.129
41	5.031	1.971	56	13.678	3.671
42	5.466	2.114	57	5.989	3.814
43	3.009	2.105	58	10.196	3.830
44	5.337	1.649	59	13.031	3.149
45	5.151	2.347	60	9.028	2.511

Structure 8 (T_s = 0.9 sec, ζ = 5%); Earthquakes (T_g = 0.25 sec)

EQ	μ_m	R	EQ	μ_m	R
1	1.459	1.0	16	1.809	1.260
2	1.140	1.0	17	1.931	1.624
3	0.990	*	18	2.210	1.518
4	1.828	1.307	19	2.307	1.390
5	1.106	1.0	20	1.653	1.317
6	1.669	1.116	21	4.638	2.173
7	1.107	1.012	22	2.464	1.633
8	0.909	*	23	3.035	1.768
9	1.768	1.149	24	3.353	1.461
10	1.491	1.092	25	2.891	1.394
11	1.643	1.200	26	3.751	1.518
12	3.205	1.393	27	2.956	1.514
13	2.408	1.789	28	2.362	1.421
14	2.006	1.370	29	4.257	2.539
15	2.401	1.497	30	6.055	1.771

Note: * indicates structure remains elastic

Structure 8 ($T_s = 0.9$ sec, $\zeta = 5\%$); Earthquakes ($T_g = 0.4$ sec)

EQ	μ_m	R	EQ	μ_m	R
31	1.411	1.287	46	8.620	2.311
32	2.282	1.301	47	3.171	1.962
33	1.213	1.163	48	3.280	1.659
34	1.819	1.226	49	3.746	2.125
35	1.995	1.235	50	2.310	1.633
36	1.519	1.261	51	14.748	2.090
37	2.379	1.426	52	13.257	1.879
38	2.200	1.244	53	16.850	2.743
39	1.856	1.276	54	8.243	2.817
40	1.892	1.325	55	15.144	1.854
41	3.652	1.612	56	12.505	2.856
42	3.387	1.656	57	5.653	2.900
43	1.910	1.766	58	6.573	3.168
44	3.710	1.604	59	7.321	2.386
45	2.122	1.816	60	7.667	2.256

Structure 8 (T_s = 0.9 sec, ζ = 5%); Earthquakes (T_g = 0.83 sec)

EQ	μ_m	R	EQ	μ_m	R
61	3.788	1.545	76	5.197	2.203
62	1.921	2.416	77	7.120	2.667
63	1.987	1.594	78	3.802	2.527
64	3.398	1.902	79	8.219	2.744
65	3.163	1.692	80	5.547	2.596
66	2.328	1.781	81	7.850	3.005
67	2.802	1.804	82	9.579	2.298
68	3.955	1.911	83	9.666	3.287
69	2.761	1.497	84	6.863	3.103
70	4.613	1.860	85	9.046	2.847
71	5.872	2.091	86	10.262	2.719
72	4.550	2.561	87	12.256	3.118
73	5.330	2.502	88	6.493	2.420
74	4.376	2.324	89	10.810	2.457
75	8.091	2.298	90	8.015	2.775

Structure 9 (T_s = 0.9 sec, ζ = 7%); Earthquakes (T_g = 0.25 sec)

EQ	μ_m	R	EQ	μ_m	R
1	1.040	1.000	16	1.367	1.138
2	0.943	*	17	1.652	1.343
3	0.843	*	18	1.701	1.291
4	1.461	1.144	19	1.802	1.262
5	0.976	*	20	1.382	1.147
6	1.479	1.000	21	3.437	1.881
7	0.901	*	22	2.145	1.420
8	0.752	*	23	2.567	1.468
9	1.400	1.000	24	2.599	1.272
10	1.005	1.000	25	2.016	1.256
11	1.110	1.000	26	3.003	1.295
12	2.513	1.228	27	2.441	1.317
13	1.807	1.578	28	1.669	1.263
14	1.944	1.189	29	4.108	2.254
15	1.893	1.293	30	5.331	1.568

Note: * indicates structure remains elastic

Structure 9 ($T_s = 0.9$ sec, $\zeta = 7\%$); Earthquakes ($T_g = 0.4$ sec)

EQ	μ_m	R	EQ	μ_m	R
31	1.402	1.151	46	4.384	2.061
32	1.757	1.093	47	3.076	1.832
33	1.062	1.000	48	2.381	1.487
34	1.223	1.110	49	3.071	1.970
35	1.464	1.029	50	2.003	1.417
36	1.619	1.159	51	10.325	1.802
37	2.139	1.364	52	11.402	1.782
38	1.767	1.177	53	10.500	2.438
39	1.297	1.065	54	6.458	2.691
40	1.369	1.139	55	13.384	1.708
41	2.492	1.465	56	8.752	2.346
42	3.011	1.379	57	6.651	2.350
43	1.677	1.624	58	5.113	2.749
44	2.655	1.589	59	5.584	1.972
45	1.771	1.505	60	7.327	2.007

Structure 10 (T_s = 1.2 sec, ζ = 3%); Earthquakes (T_g = 0.25 sec)

EQ	μ_m	R	EQ	μ_m	R
1	1.279	1.177	16	2.540	1.312
2	1.352	1.108	17	2.774	1.399
3	1.763	1.284	18	1.404	1.199
4	1.623	1.059	19	2.003	1.999
5	1.860	1.457	20	4.139	1.563
6	1.547	1.156	21	4.814	1.901
7	1.937	1.152	22	2.590	1.356
8	0.880	*	23	3.188	1.589
9	1.719	1.309	24	3.100	1.399
10	1.012	1.000	25	10.219	1.942
11	1.775	1.757	26	2.184	2.409
12	6.546	2.278	27	7.531	2.296
13	1.796	1.460	28	4.076	1.351
14	2.075	1.292	29	3.607	2.155
15	2.114	1.494	30	3.375	2.737

Note: * indicates structure remains elastic

Structure 10 (T_s = 1.2 sec, ζ = 3%); Earthquakes (T_g = 0.83 sec)

EQ	μ_m	R	EQ	μ_m	R
61	3.392	2.718	76	4.526	2.455
62	2.737	2.617	77	3.774	3.117
63	2.165	1.915	78	6.719	3.566
64	3.024	2.757	79	9.542	2.886
65	3.143	2.125	80	13.191	2.814
66	4.831	1.567	81	10.533	3.124
67	4.117	2.305	82	14.265	2.963
68	3.796	1.933	83	7.588	3.474
69	2.858	2.135	84	7.404	2.853
70	3.565	2.025	85	9.421	2.677
71	5.685	2.605	86	8.685	3.266
72	14.268	2.907	87	13.050	3.085
73	8.565	2.601	88	16.057	3.005
74	9.653	2.565	89	11.448	3.133
75	13.629	3.210	90	6.918	3.685

Structure 11 (T_s = 1.2 sec, ζ = 5%); Earthquakes (T_g = 0.4 sec)

EQ	μ_m	R	EQ	μ_m	R
31	2.246	1.254	46	4.309	1.506
32	1.590	1.269	47	2.751	1.896
33	1.221	1.002	48	2.565	1.763
34	2.156	1.347	49	3.797	1.524
35	2.502	1.286	50	2.947	1.542
36	1.846	1.514	51	3.428	1.535
37	1.278	1.263	52	5.976	2.167
38	1.452	1.077	53	4.937	1.984
39	1.953	1.167	54	4.343	3.292
40	1.864	1.462	55	3.869	2.128
41	3.613	1.781	56	6.951	1.942
42	4.019	1.519	57	3.416	2.044
43	1.924	1.711	58	5.079	2.353
44	3.652	2.250	59	4.482	2.694
45	3.575	1.442	60	6.050	2.178

Structure 12 (T_s = 1.2 sec, ζ = 7%); Earthquakes (T_g = 0.25 sec)

EQ	μ_m	R	EQ	μ_m	R
1	1.006	1.005	16	1.386	1.000
2	0.813	*	17	1.631	1.060
3	0.872	*	18	1.424	1.000
4	0.810	*	19	1.460	1.232
5	0.967	*	20	1.674	1.150
6	0.998	*	21	2.474	1.577
7	0.980	*	22	1.496	1.038
8	0.646	*	23	1.738	1.315
9	1.161	1.000	24	1.662	1.166
10	0.637	*	25	2.183	1.459
11	1.624	1.305	26	2.047	1.583
12	1.683	1.391	27	2.281	1.595
13	1.066	1.006	28	1.920	1.354
14	1.025	1.000	29	2.961	1.702
15	2.055	1.336	30	3.143	2.120

Note: * indicates structure remains elastic

Structure 12 ($T_s = 1.2$ sec, $\zeta = 7\%$); Earthquakes ($T_g = 0.83$ sec)

EQ	μ_m	R	EQ	μ_m	R
61	2.818	2.066	76	2.171	1.744
62	2.126	1.661	77	4.254	2.320
63	1.441	1.249	78	4.960	2.752
64	2.145	1.670	79	6.598	2.413
65	2.179	1.464	80	3.402	2.191
66	3.546	1.269	81	7.119	2.716
67	2.539	1.462	82	10.300	2.619
68	3.740	1.512	83	4.485	2.360
69	2.040	1.716	84	5.162	2.104
70	1.894	1.372	85	6.489	2.254
71	3.778	1.881	86	8.661	2.018
72	5.734	2.026	87	11.213	2.310
73	4.975	1.854	88	9.786	2.226
74	3.879	1.837	89	8.053	2.557
75	5.292	2.411	90	5.052	2.708

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