PB89-207187

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

State University of New York at Buffalo

# STATISTICAL EVALUATION OF RESPONSE MODIFICATION FACTORS FOR REINFORCED CONCRETE STRUCTURES

by

Howard H. M. Hwang and Jing-Wen Jaw

Center for Earthquake Research and Information

Memphis State University

Memphis, Tennessee 38152

Technical Report NCEER-89-0002

February 17, 1989

This research was conducted at Memphis State University and was partially supported by the National Science Foundation under Grant No. ECE 86-07591.

# NOTICE

This report was prepared by Memphis State University as a result of research sponsored by the National Center for Earthquake Engineering Research (NCEER). Neither NCEER, associates of NCEER, its sponsors, Memphis State University or any person acting on their behalf:

a. makes any warranty, express or implied, with respect to the use of any information, apparatus, method, or process disclosed in this report or that such use may not infringe upon privately owned rights; or

b. assumes any liabilities of whatsoever kind with respect to the use of, or the damage resulting from the use of, any information, apparatus, method or process disclosed in this report.

50272-101

| REPORT DOCUMENTATION PAGE | 1. REPORT NO. NCEER 89-0002 | 2. | 3. Recipient's Accession No. PB89-207187/AS |
|---|---|---|---|

4. Title and Subtitle
STATISTICAL EVALUATION OF RESPONSE MODIFICATION FACTORS FOR REINFORCED CONCRETE STRUCTURES

5. Report Date
February 17, 1989

6.

7. Author(s)
Howard H. M. Hwang and Jing-Wen Jaw

8. Performing Organization Rept. No.

9. Performing Organization Name and Address
National Center for Earthquake Engineering Research
State University of New York at Buffalo
Red Jacket Quadrangle
Buffalo, New York 14261

10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.
(C) 87-1004 & 88-1001
(G) ECE 86-07591

12. Sponsoring Organization Name and Address

13. Type of Report & Period Covered
Technical Report

14.

15. Supplementary Notes
This research was conducted at Memphis State University and was partially supported by the National Science Foundation under Grant No. ECE 86-07591.

16. Abstract (Limit: 200 words)
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.

17. Document Analysis a. Descriptors

b. Identifiers/Open-Ended Terms
EARTHQUAKE ENGINEERING
REINFORCED CONCRETE STRUCTURES
SOIL STRUCTURE INTERACTION
MULTIVARIATE NONLINEAR REGRESSION ANALYSIS
RESPONSE MODIFICATION FACTOR (R)
EARTHQUAKE RESISTANT DESIGN
SEISMIC DESIGN CRITERIA
BUILDING CODES
FRAMES
SHEAR WALLS

c. COSATI Field/Group

| 18. Availability Statement | 19. Security Class (This Report) | 21. No. of Pages |
|---|---|---|
| Release unlimited | Unclassified | 90 |
| | 20. Security Class (This Page) Unclassified | 22. Price A05 |

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77) (Formerly NTIS-35) Department of Commerce

# STATISTICAL EVALUATION OF RESPONSE MODIFICATION FACTORS FOR REINFORCED CONCRETE STRUCTURES

by

Howard H-M. Hwang[1] and Jing-Wen Jaw[2]

February 17, 1989

Technical Report NCEER-89-0002

NCEER Contract Number 87-1004 and 88-1001

NSF Master Contract Number ECE 86-07591

1 Associate Research Professor, Center for Earthquake Research and Information, Memphis State University

2 Research Associate, Center for Earthquake Research and Information, Memphis State University

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
State University of New York at Buffalo
Red Jacket Quadrangle, Buffalo, NY 14261

## PREFACE

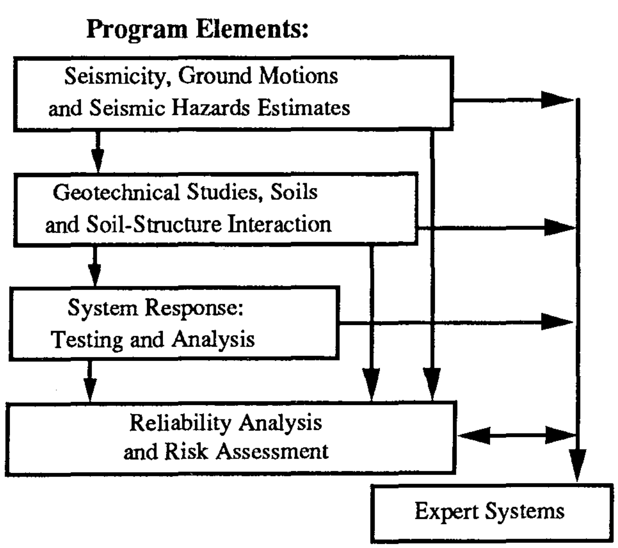
The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis 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:




**Tasks:**

Earthquake Hazards Estimates,
Ground Motion Estimates,
New Ground Motion Instrumentation,
Earthquake & Ground Motion Data Base.

Site Response Estimates,
Large Ground Deformation Estimates,
Soil-Structure Interaction.

Typical Structures and Critical Structural Components:
Testing and Analysis;
Modern Analytical Tools.

Vulnerability Analysis,
Reliability Analysis,
Risk Assessment,
Code Upgrading.

Architectural and Structural Design,
Evaluation of Existing Buildings.

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.

## TABLE OF CONTENTS





Preceding page blank

## LIST OF ILLUSTRATIONS

# LIST OF TABLES

## 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 $\omega_1$ and $\omega_2$ are the first two natural frequencies of the structure and $\zeta$ 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:

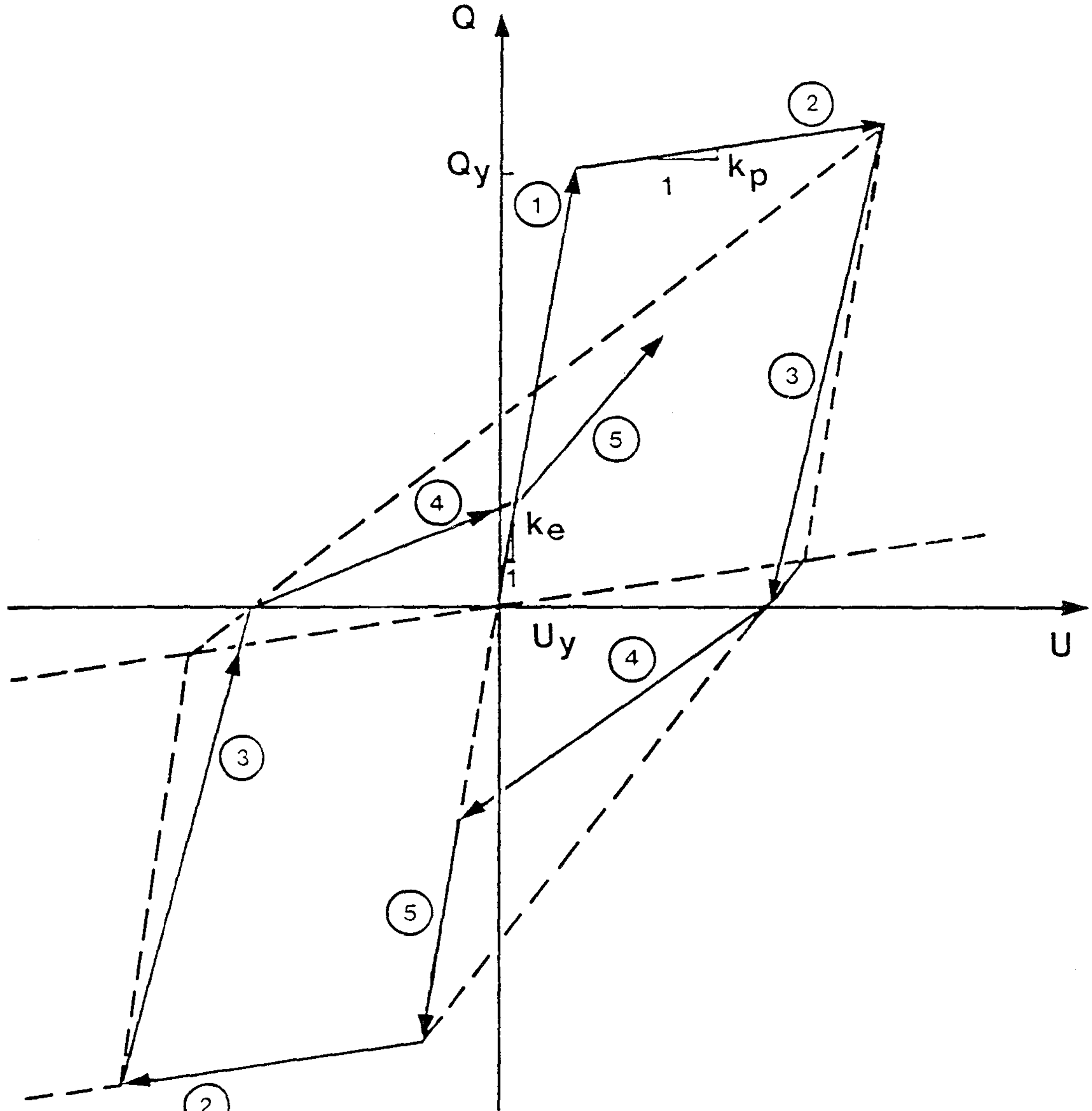



FIGURE 2-1 Hysteretic Diagram

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 $\alpha_{si}$, and pinching factor $\alpha_{pi}$. $Q_{yi}$ and $k_{ei}$ are shown in tables 3-II to 3-V. The post-yielding slope factor $\alpha_{si}$ 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 $\omega_u$ as cutoff frequency; $\omega_k = k\Delta\omega$, and $\phi_k$ = k-th random phase angle which is uniformly distributed between 0 and $2\pi$.

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]; $\omega_g$ and $\zeta_g$ 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 | $\omega_g$ (rad/sec) | $\zeta_g$ | $d_E$ (sec) |
|---|---|---|---|---|---|
| 1-10 | 0.1 | | | | |
| 11-20 | 0.15 | Rock | $8\pi$ | 0.6 | 10 |
| 21-30 | 0.2 | | | | |
| 31-40 | 0.1 | | | | |
| 41-50 | 0.15 | Deep | $5\pi$ | 0.6 | 15 |
| 51-60 | 0.2 | Cohesionless | | | |
| 61-70 | 0.1 | | | | |
| 71-80 | 0.15 | Soft | $2.4\pi$ | 0.85 | 20 |
| 81-90 | 0.2 | | | | |

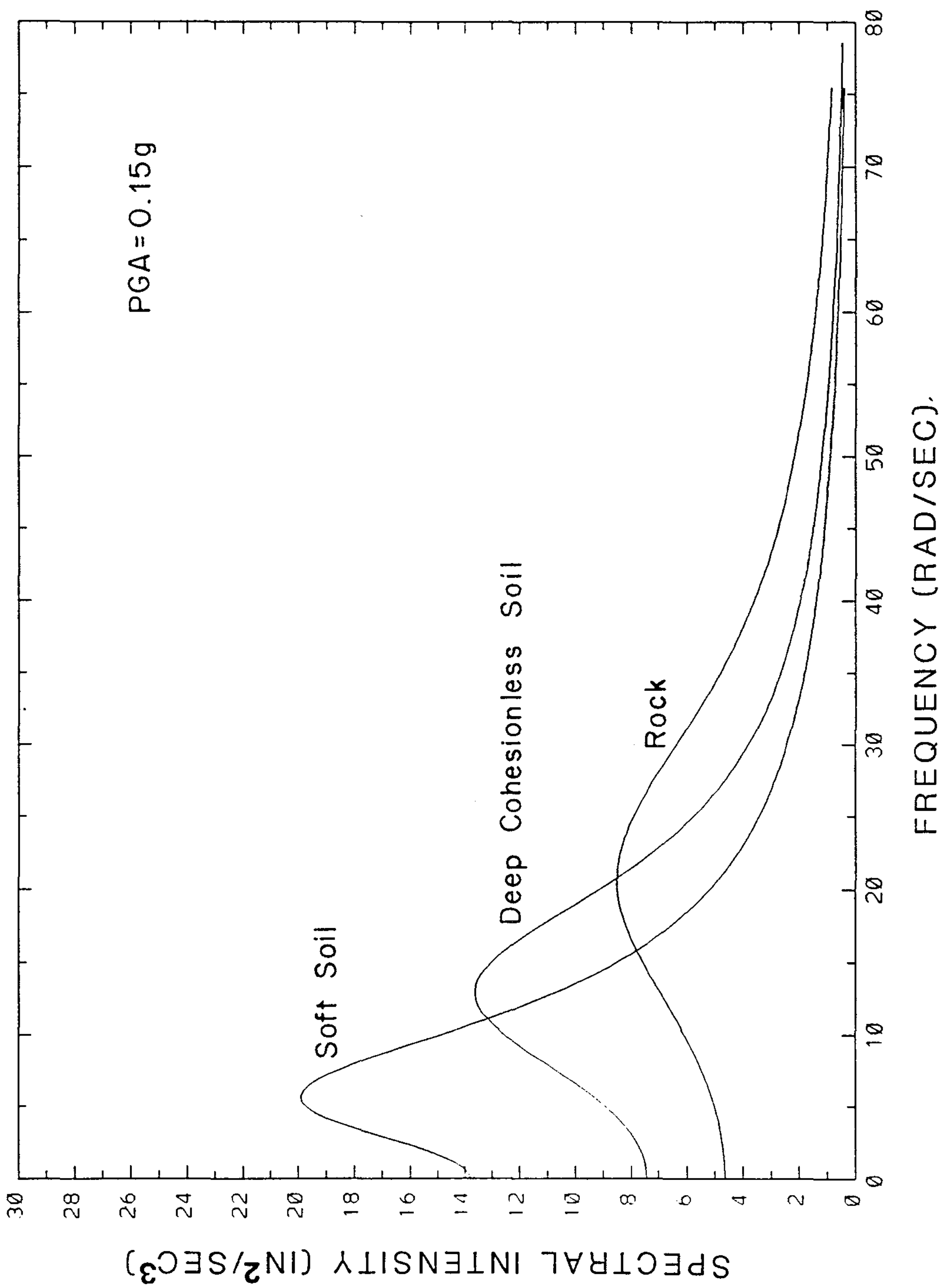



FIGURE 4-1 Power Spectra with Different Soil Conditions

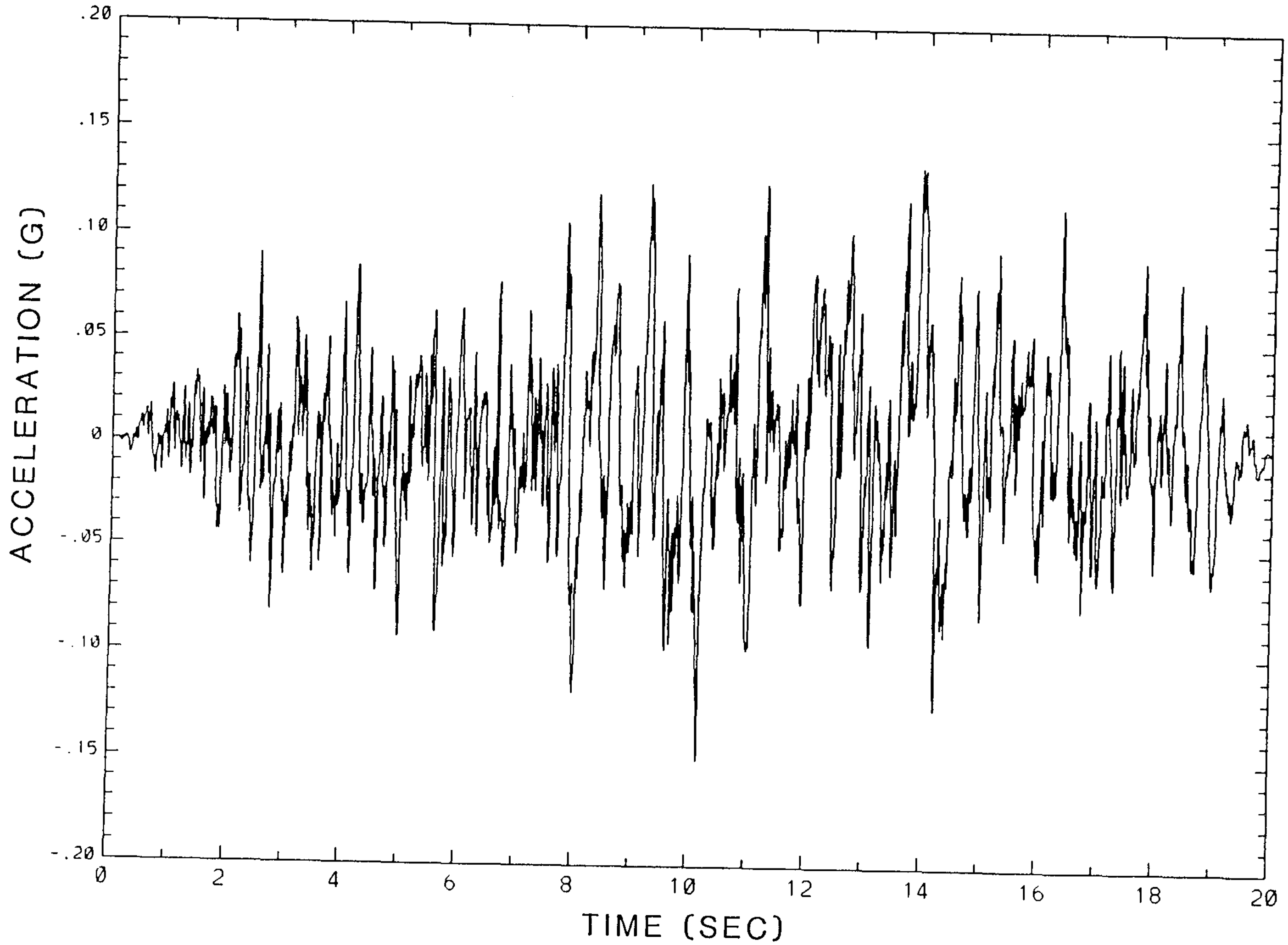



FIGURE 4-2 A Sample of Synthetic Earthquakes

# 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 $\zeta$ is the viscous damping ratio and $\mu_m$ 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 $\mu_m$ 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 $\theta_1$, $\theta_2$, and $\theta_3$ 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

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

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

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

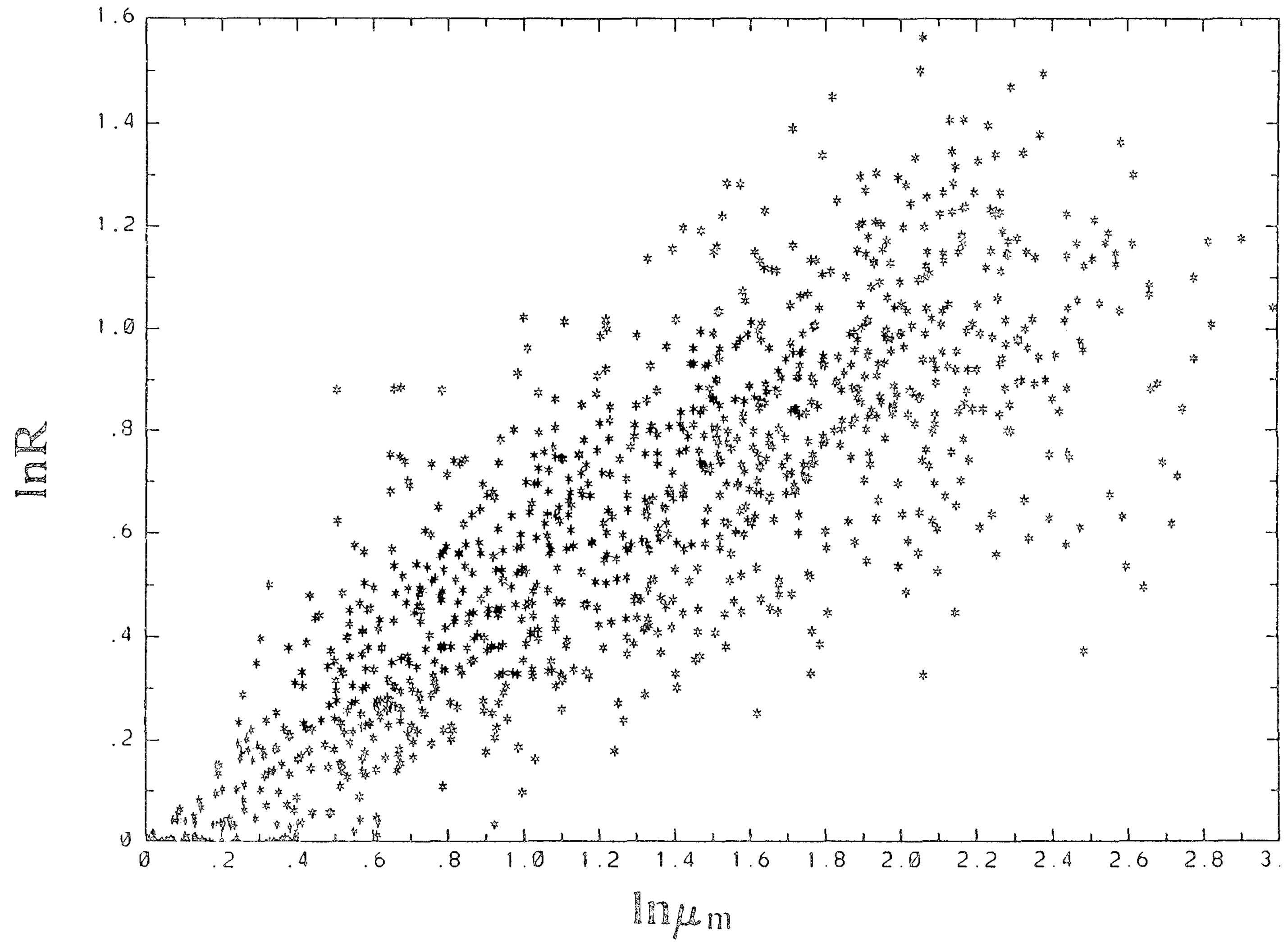



FIGURE 5-1 Scattergram of Structural Response Data

# 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 $\mu$ 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 $\mu$ and the viscous damping ratio $\zeta$ 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 $\mu_m$ 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 $\omega_g$ is taken as $2.4\pi$ rad/sec and $\zeta_g$ 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 $\mu_m$ 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.

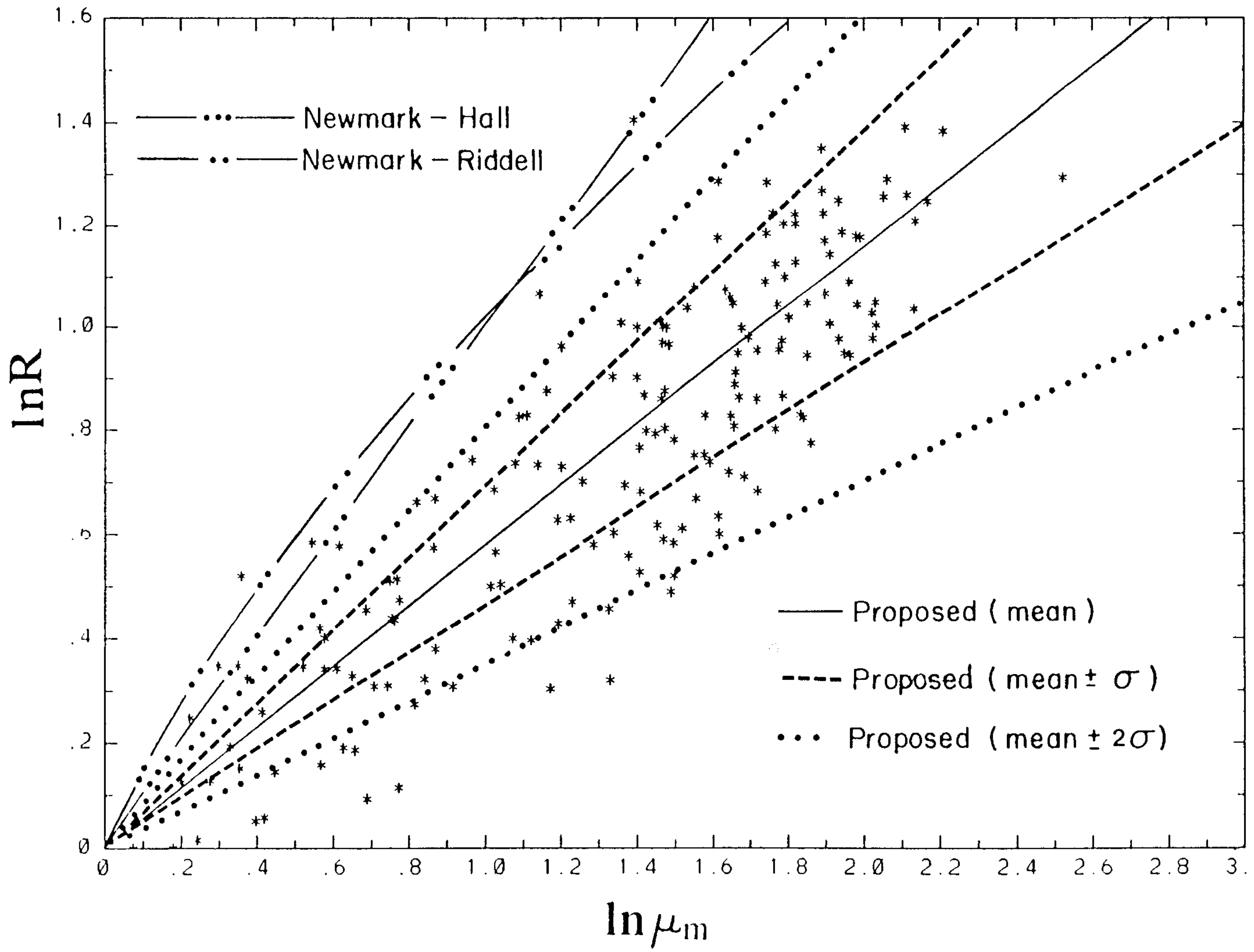



FIGURE 6-1 Comparison of Formulas for R Factor (4-Story Structure)

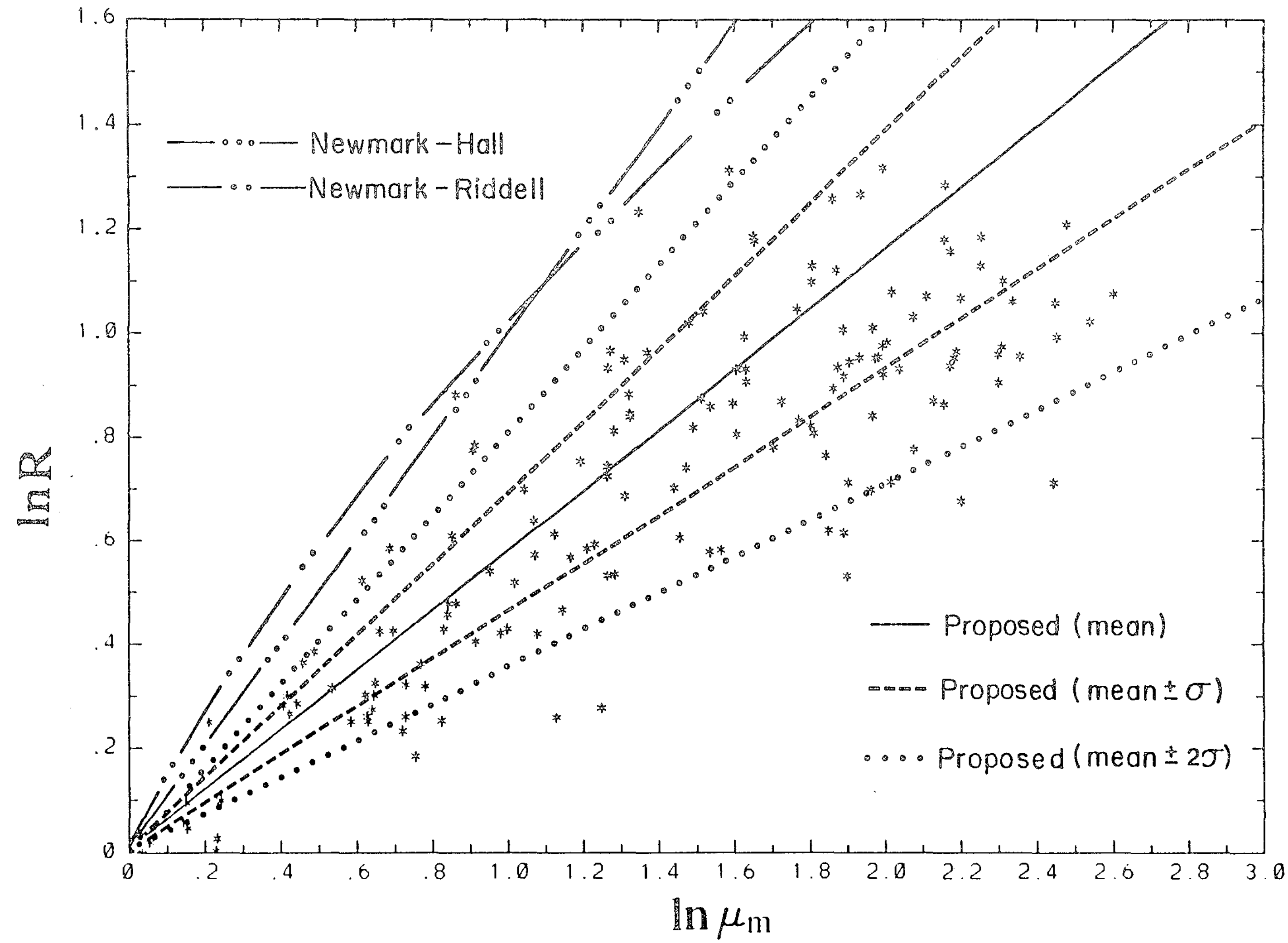



FIGURE 6-2 Comparison of Formulas for R Factor (10-Story Structure)

# 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 $\mu_m$, the viscous damping ratio $\zeta$, 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 |

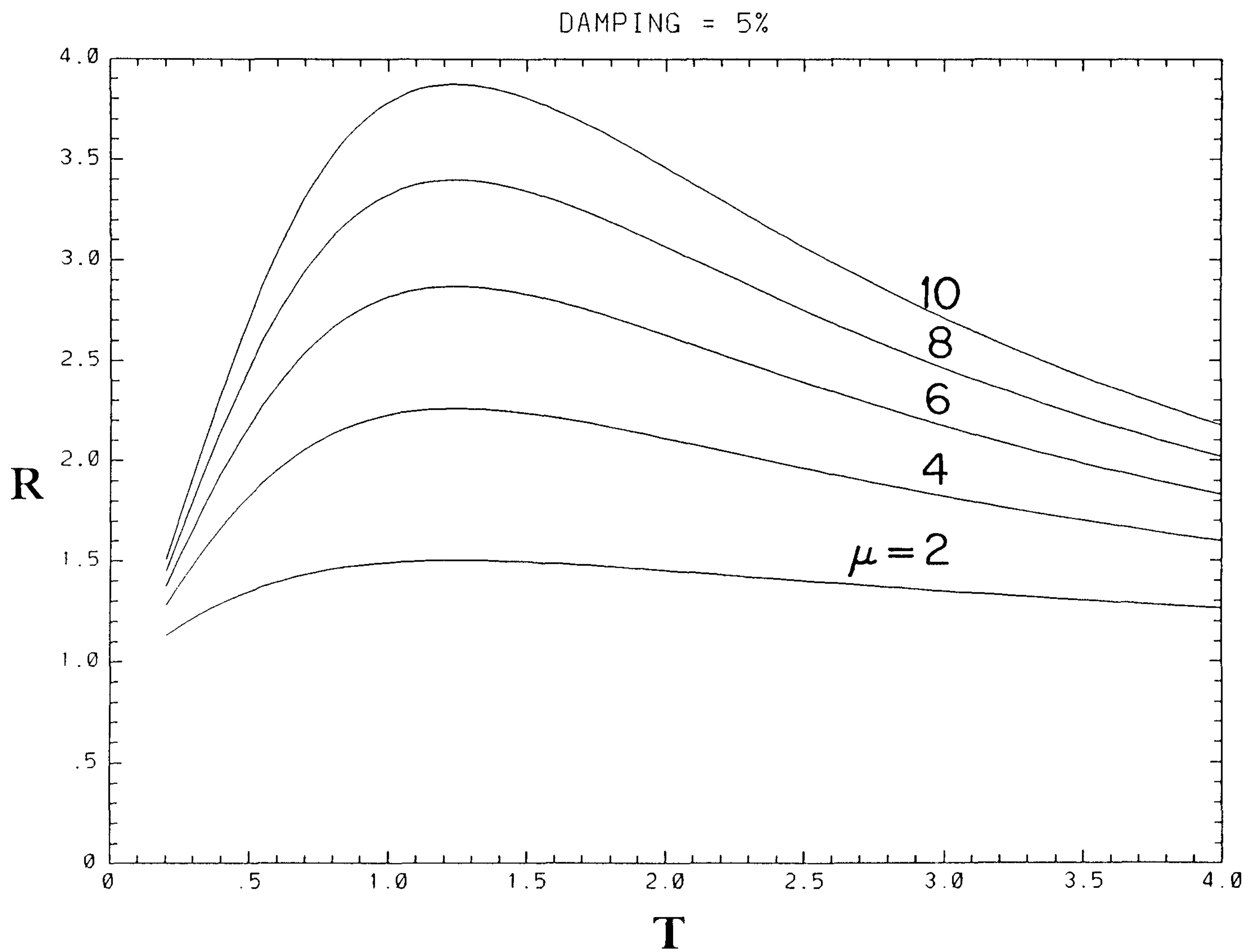



FIGURE 7-1 Recommended R Factors for Design

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
## REFERENCES


1. International Conference of Building Officials, "Uniform Building Code," 1988 Edition, Whitter, California, 1988.

2. Building Seismic Safety Council, "NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings," 1985 Edition, Washington, D.C., 1985.

3. Newmark, N.M. and Riddell, R., "Inelastic Spectra for Seismic Design," Proceedings of the Seventh World Conference on Earthquake Engineering, Istanbul, Turkey, 1980, Vol. 4, pp. 129-136.

4. Riddell, R. and Newmark, N.M., "Statistical Analysis of the Response of Nonlinear Systems Subjected to Earthquakes," Civil Engineering Studies, Structural Research Series No. 468, University of Illinois, Urbana, IL, August 1979.

5. Newmark, N.M. and Hall, W.J., "Procedures and Criteria for Earthquake Resistant Design," Building Practice for Disaster Mitigation, Building Science Series 46, National Bureau of Standards, Feb. 1973, pp. 209-236.

6. Al-Sulaimani, G.J. and Roessett, J.M., "Design Spectra for Degrading Systems," Journal of Structural Engineering, ASCE, Vol. 111, No. 12, Dec. 1985, pp. 2611-2623.

7. Hwang, H., Jaw, J.-W., and Shau, H.-J., "Seismic Performance Assessment of Code-Designed Structures," Technical Report NCEER-88-0007, National Center for Earthquake Engineering Research, SUNY, Buffalo, New York, March 1988.

8. American National Standard Institute, "Minimum Design Loads for Buildings and other Structures," ANSI A58.1-1982, New York, 1982.

9. Jaw, J.-W. and Hwang, H., "Seismic Fragility Analysis of Shear Wall Structures," Technical Report NCEER-88-0009, National Center for Earthquake Engineering Research, SUNY, Buffalo, New York, April 1988.

10. Tohma, J. and Hwang, H., "Hysteretic Model for Reinforced Concrete Containment," Transaction of the 9th International SMiRT Conference, Lausanne, Switzerland, August 1987, Vol. II, pp. 251-256.

11. Newmark, N.M., "Inelastic Design of Nuclear Reactor Structures and its Implications on Design of Critical Equipment," Transaction of the 4th International SMiRT Conference, San Francisco, August 1977, Vol. K(a),paper K4/1.

12. Shinozuka, M., "Digital Simulation of Random Process in Engineering Mechanics with the Aid of FFT Technique," Stochastic Problems in Mechanics, S.T. Ariaratnam and H.E.E. Liepholz. eds., University of Waterloo press, Waterloo, Ontario, Canada, 1974.

13. Tajimi, H., "A Statistical Method of Determining the Maximum Response of a Building Structure During an Earthquake," Proceedings of the 2nd World Conference on Earthquake Engineering, Tokyo, Vol. II, July 1960, pp. 781-798.

14. Shinozuka, M., Hwang, H., and Reich, M., "Reliability Assessment of Reinforced Concrete Containment Structures," Nuclear Engineering and Design, Vol. 80, 1984, pp. 247-267.

15. Ellingwood, B.R. and Batts, M.E., "Characterization of Earthquake Forces for Probability-based Design of Nuclear Structures," NUREG/CR-2945, U.S. Nuclear Regulatory Commission, Washington, D.C., September 1982.

16. Draper, N.R. and Smith H., "Applied Regression Analysis," John Wiley and Sons, Inc., New York, 1981.

17. "User's Manual of FORTRAN subroutines for statistical analysis," Version 1.0, International Mathematical and Statistical Libraries, Inc., Houston, Texas, April 1987.

# 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 3-1 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 |

$h_i, h_j$: height from the base to level $i$ or $j$, respectively
$W_i, W_j$: weight located at level $i$ or $j$, respectively
$n$: number of stories

The $F_t$ is determined as follows:

$$F_t = 0.07\, T_s V \leq 0.25V \quad \text{for } T_s > 0.7 \text{ sec.}$$

$$F_t = 0 \quad \text{for } T_s \leq 0.7\text{sec.} \tag{A.4}$$

From the summation of all lateral forces above the i-th floor, the story shear $Q_i$ for the i-th story is

$$Q_i = \sum_{j=i}^{n} F_j + F_t \tag{A.5}$$

Under the assumption that the story weight and story height of the structures are constant throughout the height of the structure, Eq. (A.5) can be expressed as

$$Q_i = \frac{(n-i+1)(n+i)}{n(n+1)}(V - F_t) + F_t \tag{A.6}$$

The story shears are doubled to represent story yielding strengths because the actual story strength is usually higher than the design value due to the safety factors built in the design criteria such as the load factors in load combination and the capacity reduction factors for nominal strengths. Thus, the i-th story yielding strength $Q_{yi}$ is given by

$$Q_{yi} = 2Q_i \tag{A.7}$$

# APPENDIX B

# STRUCTURAL RESPONSE DATA

**Structure 1 ($T_s$ = 0.3 sec, $\zeta$ = 3%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 3%); Earthquakes ($T_g$ = 0.4 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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 | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 5%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta = 5\%$); Earthquakes ($T_g = 0.4$ sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 5%); Earthquakes ($T_g$ = 0.83 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 7%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 7%); Earthquakes ($T_g$ = 0.4 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 7%); Earthquakes ($T_g$ = 0.83 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 3%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 3%); Earthquakes ($T_g$ = 0.4 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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 | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 5%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta = 5\%$); Earthquakes ($T_g = 0.4$ sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 5%); Earthquakes ($T_g$ = 0.83 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 7%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 7%); Earthquakes ($T_g$ = 0.4 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 7%); Earthquakes ($T_g$ = 0.83 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 3%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 1 | 1.689 | 1.239 | 16 | 2.367 | 1.687 |
| 2 | 1.540 | 1.198 | 17 | 2.896 | 2.059 |
| 3 | 1.420 | 1.103 | 18 | 2.534 | 1.837 |
| 4 | 2.262 | 1.547 | 19 | 3.645 | 1.600 |
| 5 | 1.779 | 1.192 | 20 | 2.776 | 1.592 |
| 6 | 1.767 | 1.441 | 21 | 4.555 | 2.615 |
| 7 | 2.148 | 1.365 | 22 | 2.726 | 2.011 |
| 8 | 1.182 | 1.000 | 23 | 4.537 | 2.237 |
| 9 | 2.708 | 1.425 | 24 | 3.571 | 1.672 |
| 10 | 2.574 | 1.338 | 25 | 4.690 | 1.664 |
| 11 | 2.539 | 1.557 | 26 | 8.068 | 2.008 |
| 12 | 4.382 | 1.784 | 27 | 4.037 | 1.948 |
| 13 | 2.893 | 1.974 | 28 | 3.813 | 1.664 |
| 14 | 2.688 | 1.777 | 29 | 5.540 | 3.200 |
| 15 | 3.537 | 1.815 | 30 | 6.804 | 2.084 |

**Structure 7 ($T_s$ = 0.9 sec, $\zeta$ = 3%); Earthquakes ($T_g$ = 0.4 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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 7 ($T_s$ = 0.9 sec, $\zeta$ = 3%); Earthquakes ($T_g$ = 0.83 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 61 | 3.770 | 1.901 | 76 | 10.070 | 2.658 |
| 62 | 2.710 | 2.782 | 77 | 8.196 | 3.403 |
| 63 | 1.903 | 1.977 | 78 | 11.759 | 3.209 |
| 64 | 5.318 | 2.395 | 79 | 8.599 | 3.161 |
| 65 | 4.142 | 2.232 | 80 | 7.017 | 3.334 |
| 66 | 4.353 | 2.082 | 81 | 8.970 | 3.552 |
| 67 | 3.416 | 2.331 | 82 | 11.386 | 2.763 |
| 68 | 4.936 | 2.427 | 83 | 12.683 | 3.214 |
| 69 | 4.382 | 2.077 | 84 | 8.533 | 3.729 |
| 70 | 4.513 | 2.189 | 85 | 10.285 | 3.157 |
| 71 | 16.065 | 2.563 | 86 | 9.376 | 3.433 |
| 72 | 7.918 | 3.521 | 87 | 13.214 | 3.909 |
| 73 | 5.992 | 3.024 | 88 | 7.732 | 2.990 |
| 74 | 7.405 | 2.697 | 89 | 16.682 | 3.224 |
| 75 | 11.100 | 2.580 | 90 | 7.491 | 3.600 |

**Structure 8 ($T_s$ = 0.9 sec, $\zeta$ = 5%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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 | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta = 5\%$); Earthquakes ($T_g = 0.83$ sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 7%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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 | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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 9 ($T_s = 0.9$ sec, $\zeta = 7\%$); Earthquakes ($T_g = 0.83$ sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 61 | 2.953 | 1.355 | 76 | 4.138 | 1.958 |
| 62 | 1.950 | 2.115 | 77 | 5.343 | 2.233 |
| 63 | 1.841 | 1.317 | 78 | 3.903 | 1.609 |
| 64 | 2.169 | 1.614 | 79 | 6.929 | 2.481 |
| 65 | 2.542 | 1.384 | 80 | 3.867 | 2.209 |
| 66 | 1.821 | 1.641 | 81 | 9.593 | 2.538 |
| 67 | 2.348 | 1.534 | 82 | 8.034 | 2.221 |
| 68 | 3.478 | 1.668 | 83 | 8.258 | 3.151 |
| 69 | 1.907 | 1.318 | 84 | 7.073 | 2.697 |
| 70 | 4.210 | 1.663 | 85 | 8.305 | 2.526 |
| 71 | 6.043 | 1.827 | 86 | 14.303 | 2.414 |
| 72 | 4.316 | 2.139 | 87 | 10.457 | 2.769 |
| 73 | 5.198 | 2.172 | 88 | 5.750 | 2.305 |
| 74 | 4.328 | 2.091 | 89 | 7.952 | 2.075 |
| 75 | 6.463 | 2.111 | 90 | 7.780 | 2.377 |

**Structure 10 ($T_s$ = 1.2 sec, $\zeta$ = 3%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 3%); Earthquakes ($T_g$ = 0.4 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 31 | 5.095 | 1.605 | 46 | 5.014 | 1.882 |
| 32 | 2.697 | 1.545 | 47 | 3.866 | 2.405 |
| 33 | 1.479 | 1.065 | 48 | 3.412 | 2.271 |
| 34 | 2.726 | 1.582 | 49 | 8.306 | 1.957 |
| 35 | 2.991 | 1.386 | 50 | 3.582 | 1.761 |
| 36 | 2.698 | 1.705 | 51 | 5.343 | 1.666 |
| 37 | 2.189 | 1.399 | 52 | 6.249 | 2.571 |
| 38 | 2.673 | 1.205 | 53 | 7.229 | 2.300 |
| 39 | 2.782 | 1.554 | 54 | 5.544 | 4.017 |
| 40 | 2.191 | 1.756 | 55 | 4.814 | 2.662 |
| 41 | 5.072 | 2.324 | 56 | 11.437 | 2.416 |
| 42 | 5.453 | 2.038 | 57 | 4.470 | 2.239 |
| 43 | 2.821 | 2.217 | 58 | 5.134 | 3.062 |
| 44 | 5.169 | 2.402 | 59 | 4.146 | 3.310 |
| 45 | 3.791 | 1.762 | 60 | 7.257 | 2.340 |

**Structure 10 ($T_s$ = 1.2 sec, $\zeta$ = 3%); Earthquakes ($T_g$ = 0.83 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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, $\zeta$ = 5%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 1 | 1.087 | 1.058 | 16 | 1.847 | 1.038 |
| 2 | 0.997 | * | 17 | 1.785 | 1.141 |
| 3 | 1.109 | 1.042 | 18 | 1.514 | 1.038 |
| 4 | 1.016 | 1.0 | 19 | 1.771 | 1.509 |
| 5 | 1.203 | 1.163 | 20 | 2.236 | 1.312 |
| 6 | 1.193 | 1.035 | 21 | 3.362 | 1.730 |
| 7 | 1.229 | 1.040 | 22 | 1.880 | 1.180 |
| 8 | 0.731 | * | 23 | 1.995 | 1.435 |
| 9 | 1.735 | 1.075 | 24 | 1.959 | 1.267 |
| 10 | 0.783 | * | 25 | 5.527 | 1.622 |
| 11 | 1.696 | 1.490 | 26 | 2.369 | 1.887 |
| 12 | 2.284 | 1.748 | 27 | 2.625 | 1.885 |
| 13 | 1.365 | 1.183 | 28 | 1.826 | 1.399 |
| 14 | 1.227 | 1.032 | 29 | 2.883 | 1.811 |
| 15 | 2.205 | 1.396 | 30 | 3.291 | 2.391 |

Note: * indicates structure remains elastic

**Structure 11 ($T_s$ = 1.2 sec, $\zeta$ = 5%); Earthquakes ($T_g$ = 0.4 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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 11 ($T_s$ = 1.2 sec, $\zeta$ = 5%); Earthquakes ($T_g$ = 0.83 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 61 | 3.167 | 2.339 | 76 | 2.770 | 1.928 |
| 62 | 2.463 | 1.962 | 77 | 3.961 | 2.626 |
| 63 | 1.702 | 1.489 | 78 | 5.000 | 3.157 |
| 64 | 2.214 | 2.042 | 79 | 7.834 | 2.558 |
| 65 | 2.730 | 1.692 | 80 | 7.181 | 2.328 |
| 66 | 4.058 | 1.388 | 81 | 8.798 | 2.711 |
| 67 | 3.253 | 1.787 | 82 | 11.495 | 2.830 |
| 68 | 3.957 | 1.583 | 83 | 6.524 | 2.703 |
| 69 | 2.420 | 1.908 | 84 | 6.930 | 2.288 |
| 70 | 1.777 | 1.655 | 85 | 7.726 | 2.373 |
| 71 | 4.553 | 2.174 | 86 | 9.843 | 2.339 |
| 72 | 7.021 | 2.352 | 87 | 11.944 | 2.609 |
| 73 | 8.445 | 2.134 | 88 | 10.147 | 2.455 |
| 74 | 4.830 | 2.081 | 89 | 9.744 | 2.764 |
| 75 | 6.765 | 2.767 | 90 | 5.889 | 3.108 |

**Structure 12 ($T_s$ = 1.2 sec, $\zeta$ = 7%); Earthquakes ($T_g$ = 0.25 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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 ($T_g$ = 0.4 sec)**

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 31 | 1.837 | 1.050 | 46 | 3.493 | 1.311 |
| 32 | 1.292 | 1.086 | 47 | 2.779 | 1.630 |
| 33 | 1.069 | 1.000 | 48 | 2.338 | 1.459 |
| 34 | 1.494 | 1.179 | 49 | 2.637 | 1.390 |
| 35 | 2.024 | 1.217 | 50 | 2.134 | 1.382 |
| 36 | 1.649 | 1.368 | 51 | 5.817 | 1.389 |
| 37 | 1.209 | 1.145 | 52 | 5.632 | 1.888 |
| 38 | 1.133 | 1.052 | 53 | 3.097 | 1.777 |
| 39 | 1.267 | 1.007 | 54 | 4.531 | 2.811 |
| 40 | 1.529 | 1.250 | 55 | 3.558 | 1.770 |
| 41 | 3.002 | 1.596 | 56 | 5.812 | 1.676 |
| 42 | 2.668 | 1.388 | 57 | 3.082 | 1.923 |
| 43 | 1.784 | 1.401 | 58 | 4.865 | 1.816 |
| 44 | 3.343 | 2.049 | 59 | 4.558 | 2.336 |
| 45 | 2.457 | 1.194 | 60 | 4.578 | 2.099 |

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

| EQ | $\mu_m$ | $R$ | EQ | $\mu_m$ | $R$ |
|---|---|---|---|---|---|
| 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 |

**NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH**
**LIST OF PUBLISHED TECHNICAL REPORTS**

The National Center for Earthquake Engineering Research (NCEER) publishes technical reports on a variety of subjects related to earthquake engineering written by authors funded through NCEER. These reports are available from both NCEER's Publications Department and the National Technical Information Service (NTIS). Requests for reports should be directed to the Publications Department, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Red Jacket Quadrangle, Buffalo, New York 14261. Reports can also be requested through NTIS, 5285 Port Royal Road, Springfield, Virginia 22161. NTIS accession numbers are shown in parenthesis, if available.

NCEER-87-0001 "First-Year Program in Research, Education and Technology Transfer," 3/5/87, (PB88-134275/AS).

NCEER-87-0002 "Experimental Evaluation of Instantaneous Optimal Algorithms for Structural Control," by R.C. Lin, T.T. Soong and A.M. Reinhorn, 4/20/87, (PB88-134341/AS).

NCEER-87-0003 "Experimentation Using the Earthquake Simulation Facilities at University at Buffalo," by A.M. Reinhorn and R.L. Ketter, to be published.

NCEER-87-0004 "The System Characteristics and Performance of a Shaking Table," by J.S. Hwang, K.C. Chang and G.C. Lee, 6/1/87, (PB88-134259/AS).

NCEER-87-0005 "A Finite Element Formulation for Nonlinear Viscoplastic Material Using a Q Model," by O. Gyebi and G. Dasgupta, 11/2/87, (PB88-213764/AS).

NCEER-87-0006 "Symbolic Manipulation Program (SMP) - Algebraic Codes for Two and Three Dimensional Finite Element Formulations," by X. Lee and G. Dasgupta, 11/9/87, (PB88-219522/AS).

NCEER-87-0007 "Instantaneous Optimal Control Laws for Tall Buildings Under Seismic Excitations," by J.N. Yang, A. Akbarpour and P. Ghaemmaghami, 6/10/87, (PB88-134333/AS).

NCEER-87-0008 "IDARC: Inelastic Damage Analysis of Reinforced Concrete Frame - Shear-Wall Structures," by Y.J. Park, A.M. Reinhorn and S.K. Kunnath, 7/20/87, (PB88-134325/AS).

NCEER-87-0009 "Liquefaction Potential for New York State: A Preliminary Report on Sites in Manhattan and Buffalo," by M. Budhu, V. Vijayakumar, R.F. Giese and L. Baumgras, 8/31/87, (PB88-163704/AS). This report is available only through NTIS (see address given above).

NCEER-87-0010 "Vertical and Torsional Vibration of Foundations in Inhomogeneous Media," by A.S. Veletsos and K.W. Dotson, 6/1/87, (PB88-134291/AS).

NCEER-87-0011 "Seismic Probabilistic Risk Assessment and Seismic Margins Studies for Nuclear Power Plants," by Howard H.M. Hwang, 6/15/87, (PB88-134267/AS). This report is available only through NTIS (see address given above).

NCEER-87-0012 "Parametric Studies of Frequency Response of Secondary Systems Under Ground-Acceleration Excitations," by Y. Yong and Y.K. Lin, 6/10/87, (PB88-134309/AS).

NCEER-87-0013 "Frequency Response of Secondary Systems Under Seismic Excitation," by J.A. HoLung, J. Cai and Y.K. Lin, 7/31/87, (PB88-134317/AS).

NCEER-87-0014 "Modelling Earthquake Ground Motions in Seismically Active Regions Using Parametric Time Series Methods," by G.W. Ellis and A.S. Cakmak, 8/25/87, (PB88-134283/AS).

NCEER-87-0015 "Detection and Assessment of Seismic Structural Damage," by E. DiPasquale and A.S. Cakmak, 8/25/87, (PB88-163712/AS).

NCEER-87-0016 "Pipeline Experiment at Parkfield, California," by J. Isenberg and E. Richardson, 9/15/87, (PB88-163720/AS).

NCEER-87-0017 "Digital Simulation of Seismic Ground Motion," by M. Shinozuka, G. Deodatis and T. Harada, 8/31/87, (PB88-155197/AS). This report is available only through NTIS (see address given above).

NCEER-87-0018 "Practical Considerations for Structural Control: System Uncertainty, System Time Delay and Truncation of Small Control Forces," J.N. Yang and A. Akbarpour, 8/10/87, (PB88-163738/AS).

NCEER-87-0019 "Modal Analysis of Nonclassically Damped Structural Systems Using Canonical Transformation," by J.N. Yang, S. Sarkani and F.X. Long, 9/27/87, (PB88-187851/AS).

NCEER-87-0020 "A Nonstationary Solution in Random Vibration Theory," by J.R. Red-Horse and P.D. Spanos, 11/3/87, (PB88-163746/AS).

NCEER-87-0021 "Horizontal Impedances for Radially Inhomogeneous Viscoelastic Soil Layers," by A.S. Veletsos and K.W. Dotson, 10/15/87, (PB88-150859/AS).

NCEER-87-0022 "Seismic Damage Assessment of Reinforced Concrete Members," by Y.S. Chung, C. Meyer and M. Shinozuka, 10/9/87, (PB88-150867/AS). This report is available only through NTIS (see address given above).

NCEER-87-0023 "Active Structural Control in Civil Engineering," by T.T. Soong, 11/11/87, (PB88-187778/AS).

NCEER-87-0024 Vertical and Torsional Impedances for Radially Inhomogeneous Viscoelastic Soil Layers," by K.W. Dotson and A.S. Veletsos, 12/87, (PB88-187786/AS).

NCEER-87-0025 "Proceedings from the Symposium on Seismic Hazards, Ground Motions, Soil-Liquefaction and Engineering Practice in Eastern North America," October 20-22, 1987, edited by K.H. Jacob, 12/87, (PB88-188115/AS).

NCEER-87-0026 "Report on the Whittier-Narrows, California, Earthquake of October 1, 1987," by J. Pantelic and A. Reinhorn, 11/87, (PB88-187752/AS). This report is available only through NTIS (see address given above).

NCEER-87-0027 "Design of a Modular Program for Transient Nonlinear Analysis of Large 3-D Building Structures," by S. Srivastav and J.F. Abel, 12/30/87, (PB88-187950/AS).

NCEER-87-0028 "Second-Year Program in Research, Education and Technology Transfer," 3/8/88, (PB88-219480/AS).

NCEER-88-0001 "Workshop on Seismic Computer Analysis and Design of Buildings With Interactive Graphics," by W. McGuire, J.F. Abel and C.H. Conley, 1/18/88, (PB88-187760/AS).

NCEER-88-0002 "Optimal Control of Nonlinear Flexible Structures," by J.N. Yang, F.X. Long and D. Wong, 1/22/88, (PB88-213772/AS).

NCEER-88-0003 "Substructuring Techniques in the Time Domain for Primary-Secondary Structural Systems," by G.D. Manolis and G. Juhn, 2/10/88, (PB88-213780/AS).

NCEER-88-0004 "Iterative Seismic Analysis of Primary-Secondary Systems," by A. Singhal, L.D. Lutes and P.D. Spanos, 2/23/88, (PB88-213798/AS).

NCEER-88-0005 "Stochastic Finite Element Expansion for Random Media," by P.D. Spanos and R. Ghanem, 3/14/88, (PB88-213806/AS).

NCEER-88-0006 "Combining Structural Optimization and Structural Control," by F.Y. Cheng and C.P. Pantelides, 1/10/88, (PB88-213814/AS).

NCEER-88-0007 "Seismic Performance Assessment of Code-Designed Structures," by H.H-M. Hwang, J-W. Jaw and H-J. Shau, 3/20/88, (PB88-219423/AS).

NCEER-88-0008 "Reliability Analysis of Code-Designed Structures Under Natural Hazards," by H.H-M. Hwang, H. Ushiba and M. Shinozuka, 2/29/88, (PB88-229471/AS).

NCEER-88-0009 "Seismic Fragility Analysis of Shear Wall Structures," by J-W Jaw and H.H-M. Hwang, 4/30/88, (PB89-102867/AS).

NCEER-88-0010 "Base Isolation of a Multi-Story Building Under a Harmonic Ground Motion - A Comparison of Performances of Various Systems," by F-G Fan, G. Ahmadi and I.G. Tadjbakhsh, 5/18/88, (PB89-122238/AS).

NCEER-88-0011 "Seismic Floor Response Spectra for a Combined System by Green's Functions," by F.M. Lavelle, L.A. Bergman and P.D. Spanos, 5/1/88, (PB89-102875/AS).

NCEER-88-0012 "A New Solution Technique for Randomly Excited Hysteretic Structures," by G.Q. Cai and Y.K. Lin, 5/16/88, (PB89-102883/AS).

NCEER-88-0013 "A Study of Radiation Damping and Soil-Structure Interaction Effects in the Centrifuge," by K. Weissman, supervised by J.H. Prevost, 5/24/88, (PB89-144703/AS).

NCEER-88-0014 "Parameter Identification and Implementation of a Kinematic Plasticity Model for Frictional Soils," by J.H. Prevost and D.V. Griffiths, to be published.

NCEER-88-0015 "Two- and Three- Dimensional Dynamic Finite Element Analyses of the Long Valley Dam," by D.V. Griffiths and J.H. Prevost, 6/17/88, (PB89-144711/AS).

NCEER-88-0016 "Damage Assessment of Reinforced Concrete Structures in Eastern United States," by A.M. Reinhorn, M.J. Seidel, S.K. Kunnath and Y.J. Park, 6/15/88, (PB89-122220/AS).

NCEER-88-0017 "Dynamic Compliance of Vertically Loaded Strip Foundations in Multilayered Viscoelastic Soils," by S. Ahmad and A.S.M. Israil, 6/17/88, (PB89-102891/AS).

NCEER-88-0018 "An Experimental Study of Seismic Structural Response With Added Viscoelastic Dampers," by R.C. Lin, Z. Liang, T.T. Soong and R.H. Zhang, 6/30/88, (PB89-122212/AS).

NCEER-88-0019 "Experimental Investigation of Primary - Secondary System Interaction," by G.D. Manolis, G. Juhn and A.M. Reinhorn, 5/27/88, (PB89-122204/AS).

NCEER-88-0020 "A Response Spectrum Approach For Analysis of Nonclassically Damped Structures," by J.N. Yang, S. Sarkani and F.X. Long, 4/22/88, (PB89-102909/AS).

NCEER-88-0021 "Seismic Interaction of Structures and Soils: Stochastic Approach," by A.S. Veletsos and A.M. Prasad, 7/21/88, (PB89-122196/AS).

NCEER-88-0022 "Identification of the Serviceability Limit State and Detection of Seismic Structural Damage," by E. DiPasquale and A.S. Cakmak, 6/15/88, (PB89-122188/AS).

NCEER-88-0023 "Multi-Hazard Risk Analysis: Case of a Simple Offshore Structure," by B.K. Bhartia and E.H. Vanmarcke, 7/21/88, (PB89-145213/AS).

NCEER-88-0024 "Automated Seismic Design of Reinforced Concrete Buildings," by Y.S. Chung, C. Meyer and M. Shinozuka, 7/5/88, (PB89-122170/AS).

NCEER-88-0025 "Experimental Study of Active Control of MDOF Structures Under Seismic Excitations," by L.L. Chung, R.C. Lin, T.T. Soong and A.M. Reinhorn, 7/10/88, (PB89-122600/AS).

NCEER-88-0026 "Earthquake Simulation Tests of a Low-Rise Metal Structure," by J.S. Hwang, K.C. Chang, G.C. Lee and R.L. Ketter, 8/1/88, (PB89-102917/AS).

NCEER-88-0027 "Systems Study of Urban Response and Reconstruction Due to Catastrophic Earthquakes," by F. Kozin and H.K. Zhou, 9/22/88, to be published.

NCEER-88-0028 "Seismic Fragility Analysis of Plane Frame Structures," by H.H-M. Hwang and Y.K. Low, 7/31/88, (PB89-131445/AS).

NCEER-88-0029 "Response Analysis of Stochastic Structures," by A. Kardara, C. Bucher and M. Shinozuka, 9/22/88.

NCEER-88-0030 "Nonnormal Accelerations Due to Yielding in a Primary Structure," by D.C.K. Chen and L.D. Lutes, 9/19/88, (PB89-131437/AS).

NCEER-88-0031 "Design Approaches for Soil-Structure Interaction," by A.S. Veletsos, A.M. Prasad and Y. Tang, 12/30/88.

NCEER-88-0032 "A Re-evaluation of Design Spectra for Seismic Damage Control," by C.J. Turkstra and A.G. Tallin, 11/7/88, (PB89-145221/AS).

NCEER-88-0033 "The Behavior and Design of Noncontact Lap Splices Subjected to Repeated Inelastic Tensile Loading," by V.E. Sagan, P. Gergely and R.N. White, 12/8/88.

NCEER-88-0034 "Seismic Response of Pile Foundations," by S.M. Mamoon, P.K. Banerjee and S. Ahmad, 11/1/88, (PB89-145239/AS).

NCEER-88-0035 "Modeling of R/C Building Structures With Flexible Floor Diaphragms (IDARC2)," by A.M. Reinhorn, S.K. Kunnath and N. Panahshahi, 9/7/88.

NCEER-88-0036 "Solution of the Dam-Reservoir Interaction Problem Using a Combination of FEM, BEM with Particular Integrals, Modal Analysis, and Substructuring," by C-S. Tsai, G.C. Lee and R.L. Ketter, 12/31/88.

NCEER-88-0037 "Optimal Placement of Actuators for Structural Control," by F.Y. Cheng and C.P. Pantelides, 8/15/88.

NCEER-88-0038 "Teflon Bearings in Aseismic Base Isolation: Experimental Studies and Mathematical Modeling," by A. Mokha, M.C. Constantinou and A.M. Reinhorn, 12/5/88.

NCEER-88-0039 "Seismic Behavior of Flat Slab High-Rise Buildings in the New York City Area," by P. Weidlinger and M. Ettouney, 10/15/88, to be published.

NCEER-88-0040 "Evaluation of the Earthquake Resistance of Existing Buildings in New York City," by P. Weidlinger and M. Ettouney, 10/15/88, to be published.

NCEER-88-0041 "Small-Scale Modeling Techniques for Reinforced Concrete Structures Subjected to Seismic Loads," by W. Kim, A. El-Attar and R.N. White, 11/22/88.

NCEER-88-0042 "Modeling Strong Ground Motion from Multiple Event Earthquakes," by G.W. Ellis and A.S. Cakmak, 10/15/88.

NCEER-88-0043 "Nonstationary Models of Seismic Ground Acceleration," by M. Grigoriu, S.E. Ruiz and E. Rosenblueth, 7/15/88.

NCEER-88-0044 "SARCF User's Guide: Seismic Analysis of Reinforced Concrete Frames," by Y.S. Chung, C. Meyer and M. Shinozuka, 11/9/88.

NCEER-88-0045 "First Expert Panel Meeting on Disaster Research and Planning," edited by J. Pantelic and J. Stoyle, 9/15/88.

NCEER-88-0046 "Preliminary Studies of the Effect of Degrading Infill Walls on the Nonlinear Seismic Response of Steel Frames," by C.Z. Chrysostomou, P. Gergely and J.F. Abel, 12/19/88.

NCEER-88-0047 "Reinforced Concrete Frame Component Testing Facility - Design, Construction, Instrumentation and Operation," by S.P. Pessiki, C. Conley, T. Bond, P. Gergely and R.N. White, 12/16/88.

NCEER-89-0001 "Effects of Protective Cushion and Soil Compliancy on the Response of Equipment Within a Seismically Excited Building," by J.A. HoLung, 2/16/89.

NCEER-89-0002 "Statistical Evaluation of Response Modification Factors for Reinforced Concrete Structures," by H.H-M. Hwang and J-W. Jaw, 2/17/89.