ENGINEERING RESEARCH
State University of New York at Buffalo

Evaluation of SEAOC Design Requirements for Sliding Isolated Structures

by

D. Theodossiou and M. C. Constantinou
Department of Civil Engineering
State University of New York at Buffalo
Buffalo, New York 14260

Technical Report NCEER-91-0015

June 10, 1991

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

NOTICE

This report was prepared by the State University of New York at Buffalo as a result of research sponsored by the National Center for Earthquake Engineering Research (NCEER). Neither NCEER, associates of NCEER, its sponsors, State University of New York at Buffalo, nor 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-91-0015	2.	3. PB92-114602

4. Title and Subtitle
Evaluation of SEAOC Design Requirements for Sliding Isolated Structures

5. Report Date
June 10, 1991

6.

7. Author(s)
D. Theodossiou and M.C. Constantinou

8. Performing Organization Rept. No.

9. Performing Organization Name and Address
Department of Civil Engineering
State University of New York at Buffalo
Buffalo, New York 14260

10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.
(C) ECE 86-07591
(G) BCS-8857080

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

13. Type of Report & Period Covered
Technical Report

14.

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

16. Abstract (Limit: 200 words)

The Structural Engineers Association of California (SEAOC) developed in 1990 the document "Tentative General Requirements for the Design and Construction of Seismic Isolated Structures". The SEAOC document specifies analysis procedures for seismically isolated structures, including a static and dynamic analysis procedure. Described in this report is a study that concentrated on verifying these procedures for sliding seismically isolated structures. The study involved the following: 1) evaluation of the response of sliding seismically isolated structures with stiff and flexible superstructure; 2) comparison of dynamic analysis results with the results of the static analysis procedure of SEAOC. The main conclusion reached is that a degree of conservatism exists in the SEAOC static analysis procedures. Specific cases are studied and the differences quantified.

17. Document Analysis a. Descriptors

b. Identifiers/Open-Ended Terms
EARTHQUAKE ENGINEERING. PROTECTIVE SYSTEMS. SECONDARY SYSTEMS.
SLIDING ISOLATION SYSTEMS. BASE ISOLATION SYSTEMS. BUILDING CODES.
FRICTION PENDULUM SYSTEMS (FPS). NONLINEAR ISOLATION SYSTEMS.
SEISMIC ISOLATION SYSTEMS. STATIC ANALYSIS. DYNAMIC ANALYSIS.

c. COSATI Field/Group

18. Availability Statement Release Unlimited	19. Security Class (This Report) Unclassified	21. No. of Pages 246
	20. Security Class (This Page) Unclassified	22. Price

(See ANSI–Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4–77)

Evaluation of SEAOC Design Requirements for Sliding Isolated Structures

by

D. Theodossiou[1] and M.C. Constantinou[2]

June 10, 1991

Technical Report NCEER-91-0015

NCEER Project Numbers 89-2101 and 90-2101

NSF Master Contract Number ECE 86-07591

and

NSF Grant Number BCS-8857080

1 Research Assistant, Department of Civil Engineering, State University of New York at Buffalo
2 Associate Professor, Department of Civil Engineering, State University of New York at Buffalo

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.

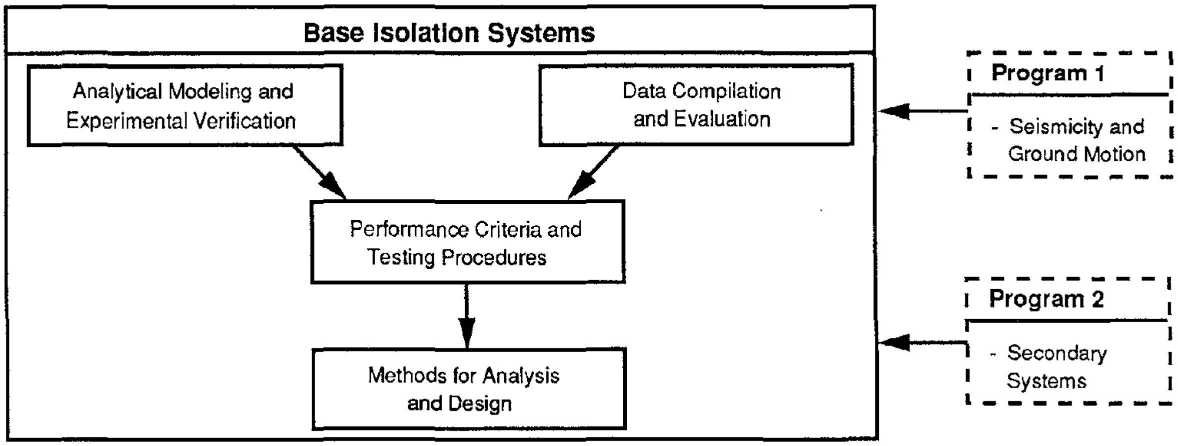
NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

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

This technical report pertains to Program 2, Secondary and Protective Systems, and more specifically, to protective systems. Protective Systems are devices or systems which, when incorporated into a structure, help to improve the structure's ability to withstand seismic or other environmental loads. These systems can be passive, such as base isolators or viscoelastic dampers; or active, such as active tendons or active mass dampers; or combined passive-active systems.

Passive protective systems constitute one of the important areas of research. Current research activities, as shown schematically in the figure below, include the following:

1. Compilation and evaluation of available data.
2. Development of comprehensive analytical models.
3. Development of performance criteria and standardized testing procedures.
4. Development of simplified, code-type methods for analysis and design.



This report addresses a recently published SEAOC document entitled ***Tentative General Requirements for the Design and Construction of Seismic Isolated Structures****, in which static and dynamic analysis procedures are specified for seismically isolated structures. Specifically, analysis procedures for sliding systems are evaluated based on either test results or dynamic nonlinear time history analysis. The main conclusion reached is that a degree of conservatism exists in the SEAOC static analysis procedures. Specific cases are studied and the differences quantified.*

ABSTRACT

The Structural Engineers Association of California (SEAOC) developed a document in 1990 entitled *"Tentative General Requirements for the Design and Construction of Seismic Isolated Structures".* The document specifies analysis procedures for seismically isolated structures, including a static and a dynamic analysis procedure.

This study concentrates on verifiying these procedures for sliding seismically isolated structures. The study involves the following:

(1) Evaluation of the response of sliding seismically isolated structures with stiff and flexible superstructure, and

(2) Comparison of dynamic analysis results to results of the static analysis procedure of SEAOC.

ACKNOWLEDGMENTS

Financial support for this work has been provided by the National Center for Earthquake Research (Contract Nos. 89-2101 and 90-2101) and the National Science Foundation (Grant BCS-8857080). This work represents a product of accumulated knowledge on the behavior of sliding isolation systems at SUNY/Buffalo which has been the result of research funded by the National Science Foundation (Grant BCS-8857080), the National Center for Earthquake Engineering Research (Contract Nos. 87-2002, 88-2002A, 89-2101 and 90-2101) and various industrial sponsors (Watson Bowman Acme Corp., MTS Systems Corp., GERB Vibration Control and Earthquake Protection Systems, Inc.).

Dr. Charles Kircher, president of Charles Kircher and Accociates, Inc. of Mountain View, California and chairman of the Base Isolation Subcommittee of the Seismology Committee of the Structural Engineers Association of Northern California, supplied the earthquake records used in this study and provided interpretation on details of the SEAOC Seismic Isolation Requirements. Dr. Kircher and Dr. Bahman Laskari, of Jack Benjamin and Associates, Inc. of Mountain View, California, reviewed the technical content of the report and performed confirmatory analysis of selected results.

TABLE OF CONTENTS

Preceding page blank

TABLE OF CONTENTS (cont'd.)

TABLE OF CONTENTS (cont'd.)

LIST OF ILLUSTRATIONS

LIST OF ILLUSTRATIONS (cont'd.)

LIST OF TABLES

TABLE TITLE PAGE

TABLE TITLE PAGE

TABLE TITLE PAGE

TABLE TITLE PAGE

SECTION 1

INTRODUCTION

The seismic isolation concept is based on the premise that a structure can be substantially decoupled from potentially damaging earthquake motions. By decoupling the structure from ground vibration, a reduction occurs in the level of response of a structure, from the level that would otherwise occur in a conventional fixed-base building.

It is intended that decoupling will be accomplished using an isolation scheme which makes the fundamental period of the isolated structure several times greater than the period of the elastic fixed-base structure above the isolation system. In this way, the fundamental period of the isolated structure shifts to a period range, where the response accelerations are much less than those at the fixed-base period.

A reduction of the acceleration response is associated, however, with an increase in the displacement of the isolation system. Control of this displacement within acceptable limits is achieved by the introduction of an energy dissipating mechanism. In this respect, isolation systems consisting primarily of elastomeric bearings have been developed.

Alternatively, isolation systems have been proposed which do not shift the fundamental period of the system but rather limit the transmission of force to the superstructure by utilizing only sliding supports. However, the lack of recentering capability in such systems may result in excessively large permanent displacements. Accordingly, sliding isolation systems with various forms of recentering devices have been developed.

Fixed-base (conventional) buildings absorb earthquake forces by inelastic response of the strutural system which lengthens the period of the system and increases its energy dissipation capacity. Inelastic response may cause building damage, both to the structural system and nonstructural components. Earthquake damage can have significant cost impacts such as repair and post earthquake disruption costs, increased earthquake insurance premiums and potential liability for losses and injuries.

The base isolation alternative reduces the forces transmitted to the structure, limiting inelastic response of the structural system and damage to the building and its contents. Isolation can provide a level of performance well beyond that of conventional buildings with potential for substantial life-cycle cost reduction.

The benefits offered by the new technology of seismic isolation, have become evident and widely accepted. Currently, several buildings and bridges in California and other countries such as Japan, New Zealand, Italy, U.S.S.R., and others have been constructed by applying the seismic isolation technology.

The familiarity and general recognition of the appealing seismic isolation advantages from the professional community and the public has led to the need to extend the implementation of this concept into a wider area of construction. Accordingly, the Seismology Committee of the Structural Engineers Association of California (SEAOC) felt that the existing provisions for the design of conventional buildings should be supplemented with design requirements developed especially for seismically isolated structures. This effort was considered necessary for the following reasons:

(1) Conventional building code were not adequate for isolated building design, and

(2) Design engineers and building officials needed special code provisions for preparing, reviewing and regulating isolated building design.

On the basis of the above, the seismic isolation concept and the criteria that would be appropriate for design and construction of seismically isolated structures have been considered by various Structural Engineers Associations of California (SEAOC) groups, since the early 1980's. Specifically, in the mid-1980's, members of the southern section of SEAOC published several papers that provide guidelines for the design of buildings with seismic isolators. In 1986, the northern section of SEAOC published *"Tentative Seismic Isolation Design Require-*

ments" (SEAONC, 1986), the first collection of design provisions for seismic isolated structures. In 1990, SEAOC recognized the need for a document that would represent a consensus opinion of all its sections. The Seismology Committee of SEAOC developed *"Tentative General Requirements for the Design and Construction of Seismic Isolated Structures"* (SEAOC, 1990a) as an appendix to supplement the 1990 *"SEAOC Recommended Lateral Force Requirements"*, also known as Blue Book (SEAOC,1990b). This appendix has been adopted by the International Conference of Building Officials and has been incorporated in the 1991 Edition of the Uniform Building Code (UBC, 1991).

It is implied that these successive efforts had a common motive, arising from the fact that seismic isolation is a relative new technology. As experience with many design-related issues increases and the results of related research become available, the design requirements for seismic isolated structures can be refined accordingly. Based on this thought, the 1990 appendix of SEAOC is considered to be prepared in keeping with the most current information and the present state of the practice of seismic isolation.

However, a lack of precise knowledge of the behavior of structures supported by all of the existing types of seismic isolation systems still exists. In anticipating the arising uncertainties, the SEAOC document, rather than addressing a specific method, provides requirements that are generally applicable to a wide range of possible isolation systems. It requires rigorous dynamic analysis for all, or virtually all the isolated buildings, but also, by providing simple formulae, it accomplishes the anticipation of the uncertainties by defining lower bound limits for the predicted response values.

In this study, an attempt is made to create a set of nonlinear dynamic analysis results which examine the behavior of structures supported by a certain type of sliding isolation system. Those results are further compared to the key design requirements provided by SEAOC and in this way, provide a basis for judging the validity and applicability of those requirements.

A certain type of isolation system, the Friction Pendulum System (Zayas et al, 1987, Mokha et al, 1990b and 1991) was used in the analyses. By introducing a highly nonlinear isolation system, the comparison is of peculiar interest, since the formulas that confine the design values provided by SEAOC are based on the assumption of a linear isolation system. The selection of the Friction Pendulum System for this study is based on the belief that it represents the best sliding isolation system. Shake table tests performed at SUNY/Buffalo have provided evidence for this (Mokha et al, 1990b and 1991, Constantinou et al 1990a and 1991). However, the obtained results are representative of the behavior of other isolation systems with similar characteristics.

SECTION 2

SEAOC DESIGN PROCEDURE FOR ISOLATED STRUCTURES

The seismic isolation concept and the criteria that would be appropriate for design and construction of isolated buildings has been considered by various Structural Engineers Association of California (SEAOC) groups, since the early 1980's. These successive efforts arise from the fact that seismic isolation is a relative new technology and as experience with many design-related issues increases and the results of related research become available, the design requirements for seismically isolated structures can be refined accordingly.

On this basis, SEAOC developed a document entitled *"Tentative General Requirements for the Design and Construction of Seismic Isolated Structures"* (SEAOC, 1990a). This document specifies design procedures for seismically isolated structures.

2.1 General Requirements

Rather than addressing a specific method on seismic isolation, the SEAOC document provides requirements that are applicable to a wide range of possible seismic isolation systems. In general, it requires that an isolation system has the following basic properties: (1) remain stable for the design displacement, (2) provides increasing resistance with increasing displacement, (3) does not degrade under repeated cyclic loading, and (4) has quantifiable engineering parameters.

Furthermore, the design requirements permit the use of either one of two different procedures for determining the design-basis seismic loads. The first procedure is intended for use on stiff buildings of regular configuration located on rock or stiff soil sites, away from active faults. This procedure uses a simple formula (similar to the seismic coefficient formula now used in conventional building design) to describe peak lateral displacement and force as a function of seismic zone, soil profile, proximity to active faults and isolated building period and damping. The second approach, which is required for all the other situations, relies on dynamic analysis procedures to determine maximum force and displacement response of the isolated building. Dynamic analysis procedures include both response spectrum (linear) analysis and time history (nonlinear) analysis. The latter is required for the design of buildings with significantly nonlinear isolation systems and/or superstructure elements.

Both the static and dynamic analysis procedures are based on the same level of seismic input and require the same level of performance from the building. The Design-Basis Earthquake load corresponds to a level of ground motion that has a 10% probability of being exceeded in a 50 year time period. For buildings not requiring a site-specific hazard analysis, the Design-Basis Earthquake spectra are defined by the ground motion spectra recommended by ATC 3-06 (ATC, 1978) and are essentially the same as those specified by the Blue Book (SEAOC, 1990b) for dynamic analysis of conventional fixed-base buildings. Those design spectra are shown in Figure 2-1. As an additional requirement, the isolation system stability must be verified by test of the maximum level of the earthquake motion that can be expected at the site. This earthquake intensity is defined as the level of ground motion that has a 10 percent probability of being exceeded in a 250 year time period.

When the first procedure is implemented according to SEAOC, limits arise which are associated with certain existing conditions, such as the soil profile type, the proximity of the structure to active faults, the seismic zone, the complexity of the configuration of the structure, etc. In those cases, a dynamic analysis approach is required by using the second procedure. If dynamic analysis is to be made, the response values must be subjected to limitations for the design. The lower bound limits are determined by applying the equations prescribed in the first procedure, where the static analysis approach exists.

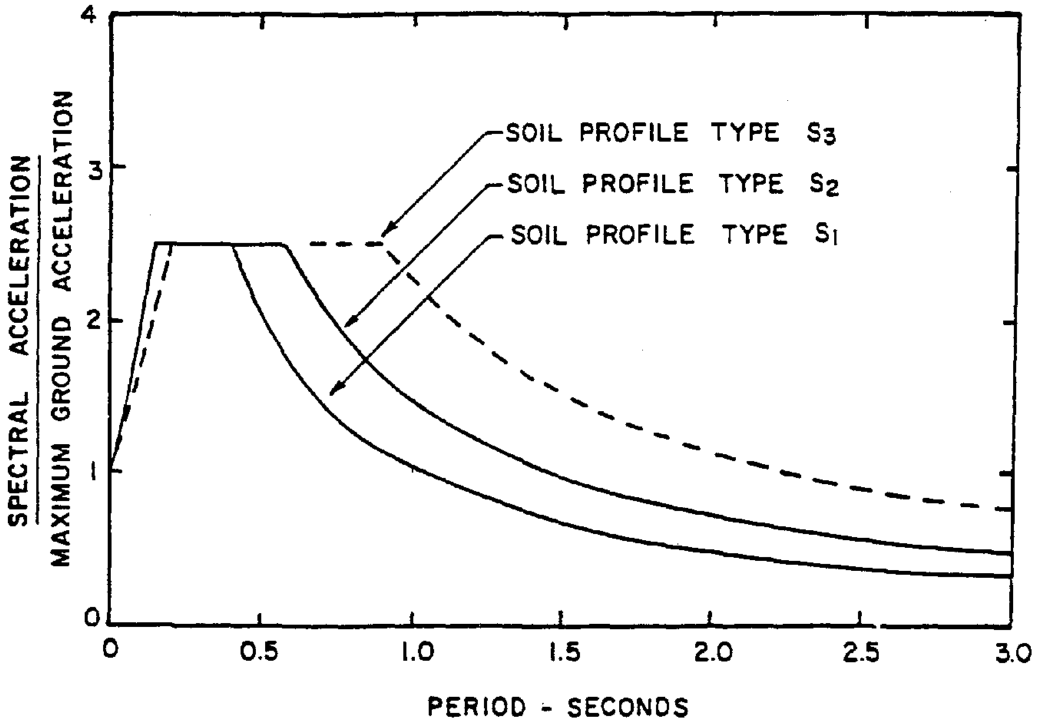


FIGURE 2-1 Normalized ATC-3 Design Spectra

Therefore, we can observe that current SEAOC thinking requires rigorous dynamic analysis for all, or virtually all, isolated buildings, but to also prescribes , by introducing a formula, a minimum design displacement of the isolation system. This approach ensures that complex or nonlinear structures will be evaluated using the appropriate dynamic analysis method and also provides a simple formula for validating the final design. The concept for using simple formula to define a lower-bound limit for isolation system displacement is analogous to the UBC's (Uniform Building Code, 1991) use of a prescriptive formula to define a minimum base shear for design of conventional fixed-base buildings. It is a process which ensures a measure of uniformity in buildings of common construction and guards against gross underdesign of key elements.

2.2 Design Methods

According to SEAOC, a seismically isolated structure may be analyzed either by the static analysis method or by a dynamic analysis method. The latter may be either a response *spectrum or a time-history* analysis. The conditions for use of these analysis methods are presented in the sequel.

2.2.1 Equivalent Static Method

2.2.1.1 Conditions for Use

According to SEAOC, a static lateral response procedure may be used for design provided:

(1) The structure is located at least 15 km from all active faults.

(2) The structure is located on a soil profile with a site factor of S1 or S2.

(3) The structure is located in Seismic Zone 3 or 4.

(4)The structure above the isolation interface is equal or less than four stories, or 65 feet, in height.

(5)The isolated period of the structure, T, is equal or less than 3.0 seconds.

(6)The isolated period of the structure, T, is greater than 3 times the elastic, fixed-base period of the structure above the Isolation Interface, as determined by the Blue Book (SEAOC, 1990b).

(7) The structure above the isolation interface is of regular configuration.

(8)The isolation system does not limit the Total Maximum Displacement to less than 1.5 times the Total Design Displacement.

(9) The isolation system is defined by all of the following attributes:

  (a) The effective stiffness of the isolation system at the Design Displacement is greater than one-third of the Effective Stiffness at 20% of the Design Displacement.

  (b) The isolation system is capable of producing a restoring force, such that the lateral force at the Total Design Displacement is at least 0.025W greater than the lateral force at 50 percent of the Total Design Displacement.

  (c) The isolation system has force deflection properties which are independent of the rate of loading.

  (d) The isolation system has force deflection properties which are independent of vertical load and bilateral load.

2.2.1.2 Design Formulae

The isolation system shall be designed and constructed to withstand minimum lateral displacements which act in the direction of each of the main horizontal axes of the structure in accordance with the formula:

$$D = \frac{10ZNST}{B} \tag{2.1}$$

where,

D = Design Displacement of the isolation system at the center of the building in the direction under consideration.

Z = Seismic Zone coefficient (Table 2.1).

N = Near field coefficient related to the proximity of the structure to active faults. (Table 2.2)

S = Soil type coefficient (Table 2.3).

T = Isolated building period, provided by equation 2.4.

B = Coefficient related to the effective damping of the isolation system (Table 2.4).

The above mentioned formula is based directly on the shape of the ATC-3 spectra for periods greater than 1.0 second, with two additional factors: (1) the near-field coefficient, N, to account for the possibility of increased displacement at sites near faults and (2) the damping coefficient, B, to account for damping in the isolation system other than 5% of critical. The relationship between this formula and the ATC-3 spectra may be seen by first setting both the damping and the near-field terms to 1.0 (i.e. 5% damped response for sites not near an active fault). Equation 2.1 becomes:

$$D = 10ZST \tag{2.2}$$

Design displacement, D, may then be converted to spectral acceleration, SA, by multiplying by $\frac{\left(2\frac{\pi}{T}\right)^2}{g}$, where g is the gravity constant (386.22 in./sec^2):

$$SA = 10ZST\frac{\left(2\frac{\pi}{T}\right)^2}{386.22} \approx \frac{ZS}{T} \tag{2.3}$$

At periods greater than 1.0 second, the above expression of spectral acceleration is seen to be consistent with the 5% design spectra recommended by the ATC-3 study for use in building codes (Figure 2-1). To use the design displacement formula (equation 2.1), an effective period of the isolated building is defined as follows:

TABLE 2.1 Seismic Zone Coefficient Z

Zone	0	1	2a	2b	3	4
Z	0.05	0.1	0.15	0.2	0.3	0.4

TABLE 2.2 Near-Field Response Coefficient N

Closest Distance, d_F, to an Active Fault	$d_F > 15$ km	$d_F = 10$ km	$d_F < 5$ km
N	1.0	1.2	1.5

TABLE 2.3 Site Coefficient S

Soil Profile Type	S1	S2	S3	S4
S	1.0	1.5	2.0	2.7

TABLE 2.4 Damping Coefficient B

Effective Damping β (Percent of Critical)	< 2%	5%	10%	20%	30%	40%	> 50%
B	0.8	1.0	1.2	1.5	1.7	1.9	2.0

$$T = 2\pi\sqrt{\frac{W}{K_{\min} g}} \tag{2.4}$$

where,

W = building weight,

g = acceleration of gravity, and

K_{min} = effective stiffness of the isolation system, determined by cyclic-load test as the slope of a line between the origin and the minimum test value of force at the design displacement.

By basing the effective period, T, on the minimum effective stiffness of the isolation system, determined by test, the above formula approximates the longest period of the isolated building at peak response. In this manner the design displacement, which is proportional to the period, is intended to prescribe the maximum excursion of the isolated structure due to the design-basis event.

The relationship between the damping coefficient, B, and the value of effective damping, β, is given in Table 2.4. The effective damping, β, is prescribed on the basis of the hysteretic behavior of the isolation system, as follows:

$$\beta = \frac{A}{2\pi K_{\max} D^2} \tag{2.5}$$

where,

A = area of the hysterisis loop determined from test results at an amplitude equal to the design displacement,

K_{max} = the maximum effective stiffness of the isolation system, determined by cyclic-load tests.

By basing the damping coefficient, B, on the force-deflection behavior of the isolation system, the above formula estimates the reduction in displacement response for systems which have damping values greater than 5% of critical.

The provisions also prescribe Total Design Displacement, D_T (Design Displacement including torsional effects), on the basis of structure configuration and eccentricity as follows:

$$D_T = D[1 + \frac{12ye}{b^2 + d^2}] \tag{2.6}$$

where,

D_T =Total Design Displacement of the isolation system, including both the translational displacement D, at the center of rigidity and the component of torsional displacement in the direction under consideration.

y = distance between the center of isolation system rigidity and the point of interest, measured perpendicular to the direction under consideration.

e = actual eccentricity between the center of mass of the structure and the center of rigidity of the isolation system, plus accidental eccentricity taken as 5% of d.

d = longest plan dimension of the structure.

b = shortest plan dimension of the structure, measured perpendicular to d.

The additional component of displacement due to torsion, as prescribed by equation 2.6, increases the Design Displacement at the corner of the structure by about 15% (for a perfectly square building in plan) to about 30% (for a very long rectangular building for an eccentricity of 5 percent). Values less than those of equation 2.6 can be used with justification, but D_T cannot be less than 1.1 D.

The Total Maximum Displacement, D_{TM} required for verification of the isolation system stability in the most critical direction of horizontal response is calculated as follows:

$$D_{TM} = 1.5D_T \tag{2.7}$$

The Total Maximum Displacement, according to SEAOC, is used to verify adequate clearances and separations, verification of isolator stability and load testing of the isolator prototypes.

The peak force below and at the isolation system, V_b, corresponding to the peak displacement D, is given by the following expression:

$$V_b = \frac{K_{max} D}{1.5} \tag{2.8a}$$

In defining this equation, SEAOC introduces a reduction factor of the order of 1.5, as to adjust the peak shear to a level compatible with the working-stress allowables specified in other sections of the Blue Book (SEAOC, 1990b).

The value of V_b shall not be taken as less than the following:

(1) The lateral seismic force required for a fixed-base structure of the same weight and a period equal to the isolated period, T.

(2) The lateral seismic force required to fully activate the isolation system (e.g. the yield level of a softening system, the ultimate capacity of a sacrificial Wind-Restraint System or the static friction level of a sliding system).

Furthermore, SEAOC specifies a minimum design force for the structure above the isolation system which is in the same form as equation 2.8a but with a reduction factor R_{WI}, other than 1.5

$$V_S = \frac{K_{max} D}{R_{WI}} \tag{2.8b}$$

This reduction factor depends on the lateral force resisting system of the superstructure and is about four times less than the R_W factor used in the calculation of the lateral force for the design of conventional non-isolated structures (SEAOC, 1990b). In this respect, it is required by SEAOC that base isolated structures remain essentially elastic for the Design Basis Earthquake.

The distribution of force V_S with height is based on an assumed uniform distribution of seismic acceleration over the height of the structure above the isolation interface.

2.2.2 Dynamic Lateral Response Procedure

2.2.2.1 Conditions for Use

According to SEAOC, a dynamic analysis, interpreted either as a response spectrum analysis or a time history analysis, is required for design of the following isolated structures:

(1) Structures with a seismic isolated period T, which is less than 3 times the elastic, fixed-base period of the structure above the isolation interface.

(2)Structures (above the isolation interface) having a stiffness, weight or geometrical vertical irregularity or other irregularity, for which the 1990 Blue Book requires dynamic analysis.

(3)Structures located within 15 km of an active fault.

(4)Structures located at a soil profile with a site factor of S3 or S4 (soft and very soft soil types).

(5) The structure is located in Seismic Zone 0,1,2A or 2B.

(6)The structure above the isolation system is greater than 4 stories or greater than 65 feet in height.

(7)The isolated period of the structure, T, is greater than 3.0 seconds.

Additionally, time history analysis is required to determine the design displacement of the isolation system and the peak floor displacements of the structure above the isolation system for the following isolated structures:

(1) The structure is located on a soil profile with a site factor S4.

(2)The isolation system limits the Maximum Credible Earthquake displacement to less than 1.5 times the Design-Basis Earthquake displacement.

(3)The isolation system has one or more of the following attributes:

(a)The Effective Stiffness at the Design Displacement is less than one-third of the Effective Stiffness at 20% of the Design Displacement.

(b)The isolation system is not capable of producing a restoring force specified in the detailed requirements.

(c)The isolation system has force deflection properties which are dependent of the rate of loading.

(d)The isolation system has force deflection properties which are dependent of vertical load and bilateral load.

2.2.2.2 Ground Motion Design Spectra

Properly substantiated, site specific spectra are required for design of all structures with an isolated period, T, greater than 3.0 seconds, or located on a soil type profile of S3, or S4, or located within 15 km of an active fault or located in Seismic Zones 1, 2A or 2B. Structures not requiring site-specific spectra shall be designed by using the spectra of Figure 2-1.

A design spectrum shall be constructed for the Design-Basis Earthquake. This design spectrum shall not be taken as less than the normalized response spectrum given in Figure 2-1 for the appropriate soil type, scaled by the seismic zone coefficient.

Exception: If a site-specific spectrum is calculated for the Design-Basis Earthquake, then the spectrum may be taken as less than 100 percent, but not less than 80 percent of the normalized response spectrum given in Figure 2-1 for the appropriate soil type, scaled by the seismic zone coefficient.

Also, a design spectrum shall be constructed for the Maximum Credible Earthquake. This design spectrum shall not be taken as less than 1.25 times the Design-Basis Earthquake spectrum. This design spectrum shall be used to determine the Total Maximum Displacement for testing of the stability of the base-isolation system.

2.2.2.3 Time Histories

Pairs of horizontal ground motion time history components shall be selected from not less than three recorded events. Each pair of time histories shall be applied simultaneously to the structure, considering the least advantageous location of the mass center. These motions shall be scaled such that the square root of the sum of the squares (SRSS) of the 5%

damped-spectrum of the scaled horizontal components does not fall below 1.3 times the 5%-damped spectrum of the Design-Basis Earthquake by more than 10% in the period range T, as described by formula 2.1, for periods from T, minus 1.0 seconds to T plus 2.0 seconds. The maximum response of the parameter of interest as calculated by the three time history analyses shall be used for design.

The duration of the time histories shall be consistent with the magnitude and source characteristics of the Design-Basis Earthquake. Time histories developed for sites 15 km of a major active fault shall incorporate near fault phenomena.

2.2.2.4 Response Spectrum Analysis

Response spectrum analysis shall be performed using a damping value equal to the effective damping of the isolation system or 30 percent of critical, whichever is less. Response spectrum analysis used to determine the Total Design Displacement and the Total Maximum Displacement shall include simultaneous excitation of the model by 100 percent of the most critical excitation of ground motion and 30 percent of the ground motion on the orthogonal axis.

2.2.3 Lower Bound Limits on Applying the Results of a Dynamic Analysis Procedure

As previously stated, certain limits confine the implementation of response values for design, if they are predicted according to a dynamic analysis procedure. These limits are described below.

2.2.3.1 Isolation System and Structural Elements Below the Isolation Interface

(1)The Total Design Displacement of the isolation system shall not be taken as less than 90 percent of D_T, as specified in the equation 2.6.

(2)The Total Maximum Displacement of the isolation system shall not be taken as less than 80% of D_{TM}, prescribed by equation 2.7.

(3)The design lateral shear force on the isolation system and structural elements below the isolation interface shall not be taken as less than 90 percent of V_b, prescribed by equation 2.8a.

2.2.3.2 Structural Elements Above the Isolation Interface

(1)The design lateral shear force on the structure above the isolation interface, if regular in configuration, shall not be taken as less than 80 percent of V_S (equation 2.8b), nor less than the limits imposed to V_S as prescribed in the Equivalent Static Method.

Exception : The design lateral shear force on the structure above the isolation interface, if regular in configuration, may be taken as less than 80 percent of V_S, but not less than 60% of V_S, provided time history analysis is used for design of the structure.

(2)The design lateral shear force on the structure above the isolation interface, if irregular in configuration, shall not be taken as less than V_S (equation 2.8b).

2.3 Application to Sliding Systems

The design methods of SEAOC, as described in Section 2.2, refer generally to all isolation systems. In interpreting the design formulae for a certain isolation system, one has to account for those parameters that configure the relation between the displacement and the developed force, for the specific isolation system.

In the case of sliding isolation systems, complications occur between the displacement of the system and the developed forces. They are attributed to the variation of the coefficient of friction with respect to the velocity of sliding. For the Friction Pendulum System (FPS), the force developed is equal to the combination of the mobilized frictional force and the restoring force which develops as a result of the induced rising of the structure along the spherical surface. (For further details on the FPS, see Section 4.1.) Figure 2-2 shows the ideal force-displacement loop for the Friction Pendulum System. The dotted extended lines describe the behavior of the FPS when the coefficient of friction has a constant value, fmax, independent of the velocity of sliding. The effective stiffness of the isolation system is then defined as follows:

$$K_{eff} = f_{\max}\frac{W}{D} + \frac{W}{R} \tag{2.9}$$

where,

f_{max} = maximum value of the coefficient of friction at high velocity of sliding.

W = weight of the structure.

D = maximum displacement of the isolation system.

R = radius of curvature of the FPS bearings.

The effective damping, β, is evaluated from equation 2.5 with the area A being:

$$A = 4Df_{\max}W \tag{2.10}$$

By substituting equation 2.10 to 2.5 and setting the value of K_{eff} equal to K_{max}, we have:

$$\beta = \frac{2}{\pi}\left(\frac{f_{\max}}{f_{\max} + \frac{D}{R}}\right) \tag{2.11}$$

The period T of the isolation system is provided by equation 2.4, which for K_{eff} equal to K_{min} and after using equation 2.9 yields:

$$T = 2\pi\sqrt{\frac{1}{f_{\max}\frac{g}{D} + \frac{g}{R}}} \tag{2.12}$$

The design displacement for the Friction Pendulum System is then calculated by equation 2.1, where the terms B and T are estimated by an iteration procedure through equations 2.11 and 2.12 and Table 2.4. The base shear is finally estimated by combining equations 2.8a and 2.9 (1.5 reduction factor is omitted):

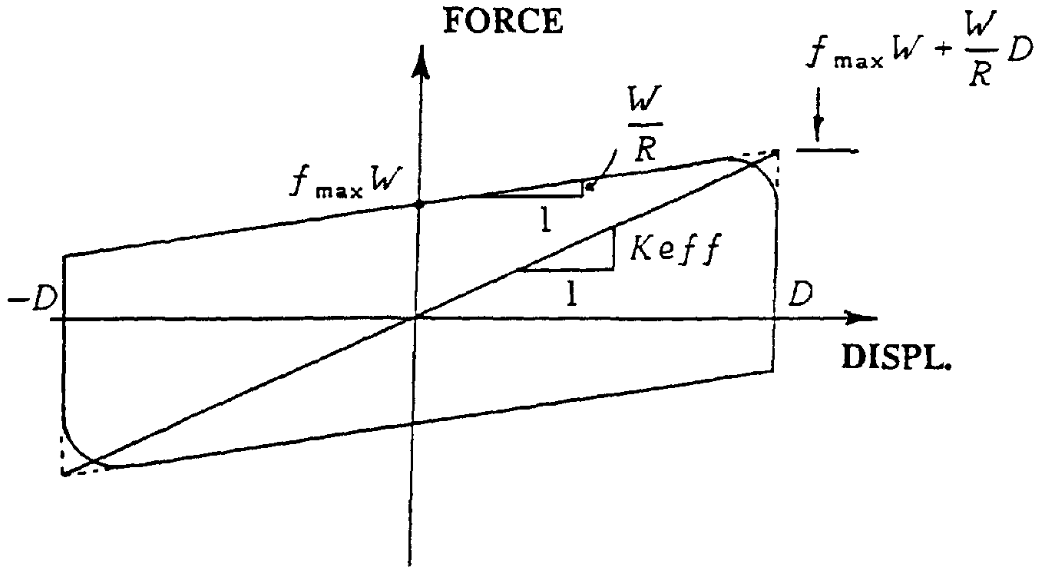


FIGURE 2-2 Ideal force-displacement loop for the Friction Pendulum System.

$$\frac{V_b}{W} = f_{\max} + \frac{D}{R} \qquad (2.13)$$

The structure shear, V_S is again given by equation 2.13 (without the R_{WI} factor). It should be noted that in developing the above equations, it has been assumed that the effective stiffness, K_{eff}, does not change between a minimum, K_{min}, and a maximum, K_{max}, value. In fact, this behavior has been observed in tests (Mokha et al, 1990b and 1991), in which the stiffness properties of FPS bearings remained unchanged under repeated testing.

SECTION 3

TECHNICAL APPROACH - OVERVIEW

In the previous section, the requirements specified by SEAOC to estimate the design values for an isolation system were introduced. According to SEAOC, a rigorous dynamic analysis is required for all, or virtually all the isolated buildings, but the predicted response values should be subjected to certain limitations, also specified by SEAOC.

The lack of response data have prompted analyses to verify the SEAOC design requirements. Specifically, the work by Kircher and Lashkari, 1989, has been reported. In their report, an evaluation of SEAOC requirements was attempted through a series of non-linear dynamic analyses of isolated structures. These analyses examined the behavior of a rigid structure supported by bearings which exhibited bilinear hysteretic behavior and was excited by various earthquake motions. Statistical quantities, such as the mean and the standard deviation values, provided a measure of the level and inherent variation of response parameters as a function of the variation in the ground motion. In this way, a basis for judging the validity and applicability of SEAOC design requirements was provided.

A similar approach is attempted in this report. By concentrating on a class of sliding seismically isolated structures, and performing dynamic analyses under various considerations, a set of nonlinear response data was obtained. This data enabled comparisons and conclusions to be developed for the SEAOC procedures.

The adopted isolation system for this study is the Friction Pendulum System (Zayas et al, 1987, Mokha et al, 1990b and 1991). This is a sliding isolation system where nonlinearity between the displacements and the developed forces exists. The performed analyses accounted for the nonlinear behavior of this isolation system. The dependence of the coefficient of friction at the frictional interface to the velocity of sliding and the bi-directional interaction of the forces at each isolation bearing, all properties of sliding isolation systems, were also taken into account. In this way, it was indicated that the performed analyses were accurate enough for the prediction of response of the examined cases.

In this study, two isolated structure models were used, which were considered to represent buildings exhibiting stiff and flexible superstructure behavior. Two structures were selected: 1 - story and an 8 - story. A mass eccentricity of the order of 5% was decided. These also provide the investigation of the potential of torsional response of the base of the isolated structure, as a function of the floors that the structure carries. Further, the increased values of the displacements due to torsional effects are compared to equation 2.6 that SEAOC introduces for defining the Total Design Displacement, accounting for torsion.

A total of three different designs of the isolation system was examined in this study and the differentiation between them is based on their properties such as the maximum coefficient of friction, f_{max}, and the radius of curvature, R, of the concave sliding surface. An analytical description of the FPS system and the values for the above properties selected for this study is made in Section 4. The combining of different values for f_{max} and R (see Section 4.2.2) was done to create a group of designs (a total of three) with combined strong or weak frictional force and restoring force. In this way, two different FPS systems always had a common value, which could be either their maximum coefficient of friction or their radius of curvature. This provides the reader with the ability to further compare the dynamic analysis results with the SEAOC procedures, and also the dynamic analysis results between them.

Four different approaches were adopted in this study for the evaluation of SEAOC design code requirements. The first one, which is developed in Section 5, proceeds to the comparison of SEAOC static procedure to response values recorded during shake table tests (extrapolated to prototype scale). In this way, the basis for judging SEAOC requirements is provided through directly recorded response values of structures as if they were excited with real earthquake motions.

All the following approaches rely on a dynamic analysis procedure, which accounts for the properties of the FPS, as described above. The earthquake motions were all generated or scaled (from real records) to correspond to Seismic Zone 4. This zone was selected since it is the zone of highest seismicity and is where most isolated structures have been or will be constructed.

The second approach (see Section 6) utilizes artificially generated earthquake motions and examines the behavior of the structures under this excitation. The records were generated to be compatible with the design spectra specified in SEAOC. In this way, earthquake motions were also obtained to correspond to soil type S3 (soft soil sites), since no real records for those sites were currently available. For every one of the site conditions under interest (S1, S2, S3 soil types), three corresponding earthquake motions were generated. In this way, for every combination of isolation system, superstructure and soil site, three identical analyses were performed which examine the behavior of the structure under excitation.

The third approach is based on the work done by Kircher and Lashkari, 1989 described in the beginning of this section. The basis for providing a statistical evaluation analysis was to examine the same structure - isolation system model under excitation with different earthquake motions that all corresponded to the same soil site. Under these conditions, statistical quantities like the mean and the standard deviation values were calculated and reported. The scaling of the records was initially based on both Peak Ground Acceleration (PGA) and Peak Ground Velocity (PGV). As explained in Section 7, the study showed that scaling according to PGA was not appropriate and reliable for use in the analyses. Accordingly, the effort was concentrated only in the case of scaling based on PGV.

The fourth approach (Section 8) implements SEAOC procedures when dynamic analysis is to be made. The earthquake motions are selected and scaled among those used in the statistical evaluation approach. A total of six pairs of scaled earthquake motions was applied and examined.

The intent of this study is to create a set of data from which comparisons and evaluations can be made for SEAOC requirements for sliding isolated structures. By selecting four different, and all acceptable approaches to the problem, the predicted results, comparisons and conclusions contain a desirable and sufficient level of reliability and validity.

SECTION 4

STRUCTURE AND ISOLATION SYSTEM MODELS

The major intent of this study is to create, using a series of nonlinear analyses, a collection of results upon which comparisons and evaluations can be made on the response of sliding isolated structures with rigid and flexible superstructure. In this work, a certain type of sliding isolation system, the Friction Pendulum System (FPS) is examined in correlation with two different types of superstructure models. These models were configured appropriately, and represent isolated buildings that exhibit a relatively rigid and flexible superstructure behavior.

The nonlinear analyses were performed using the 3D-BASIS program (Nagarajaiah et al, 1989). This program utilizes special modeling options that account for the behavior of sliding isolation systems.

4.1 Superstructure Configuration

The structure - isolation system models used in this study were representative of one and eight story moment resisting frames of rectangular configuration. In both cases, the superstructure consisted in plan of four bays by eight bays with each bay measuring 20 feet by 20 feet. One FPS isolator was placed at the intersection of the bays, for a total of 45 isolators. Floor height was 12 feet.

The one - story superstructure had a fundamental period of 0.2 seconds, whereas the eight - story superstructure had a period of 1.14 seconds. These values are representative of moment resisting frames. The weight of each floor was 1280 kips (based on a combined dead and seismic live load of 100 psf). The distribution of mass on each floor was assumed to be asymmetric so that an eccentricity of the order of 5% of the longest plan dimension was created in the longitudinal direction. Each story had identical stiffnesses in the two orthogonal directions. In the eight - story building, the first three stories had the same stiffness, the next three had 0.75 times the stiffness of the first three stories, and the last two had half the stiffness of the first three stories. The distribution of stiffnesses to the various story elements was selected in such way as to result in a torsional period in the absence of eccentricities of 0.58 times the translational period. The superstructure stiffness matrix was constructed in a shear type representation with a diagonal mass matrix.

The properties of each structural system are summarized in Tables 4.1 and 4.2. The dynamic characteristics, frequencies and mode shapes of the two systems are presented in Tables 4.3 and 4.4. It should be noted that the fundamental period of the two structures is slightly different than 0.2 and 1.14 secs, respectively. This is caused by the mass eccentricity. In the dynamic analysis of the 8 - story structure, twelve out of twenty four modes were accounted for. The other twelve modes corresponded to periods less than 0.16 seconds and their contribution was assumed insignificant.

4.2 Isolation System

4.2.1 Description

Sliding isolation systems utilize sliding interfaces (usually Teflon - steel interfaces) to support the weight of the structure. These interfaces provide little resistance to lateral loading by virtue of their low friction. Recentering capability is provided by a separate mechanism.

In the case of the FPS, the isolated structure is supported by bearings, each one consisting of an articulated slider on a spherical concave surface. A typical section of an FPS bearing is shown in Figure 4-1. The slider is faced with a bearing material which, when in contact with the polished metal spherical surface results in a maximum coefficient on friction on the order of 0.1 or less at high velocity of sliding and a minimum friction coefficient of the order of 0.05 or less at very slow velocity of sliding. This dependency of the coefficient of

TABLE 4.1 Properties for 1 - story isolated structure.

Story / Floor	Weight (kips)	Rotational Inertia (kips-in-sec^2)	Stiffness (kips/in)	Rotational Stiffness (kips-inch) ×1000	Eccentricity (ft)	
					Longitudinal	Transverse
1	1280	1272642.5	3270.958	3733792.62	8	0
Base	1280	1272642.5			8	0

TABLE 4.2 Properties for 8 - story isolated structure.

Story / Floor	Weight (kips)	Rotational Inertia (kips-in-sec^2)	Stiffness (kips/in)	Rotational Stiffness (kips-inch) ×1000	Eccentricity (ft)	
					Longitudinal	Transverse
8	1280	1272642.5	1700.898	1997933.76	8	0
7	1280	1272642.5	1700.898	1997933.76	8	0
6	1280	1272642.5	2551.347	2996900.64	8	0
5	1280	1272642.5	2551.347	2996900.64	8	0
4	1280	1272642.5	2551.347	2996900.64	8	0
3	1280	1272642.5	3401.796	3995867.52	8	0
2	1280	1272642.5	3401.796	3995867.52	8	0
1	1280	1272642.5	3401.796	3995867.52	8	0
Base	1280	1272642.5			8	0

TABLE 4.3 Dynamic characteristics of 1 - story superstructure (including 5% mass eccentricity).

Floor	Mode								
	1			2			3		
	L Component	T Component	Rotational Component	L Component	T Component	Rotational Component	L Component	T Component	Rotational Component
1	0.000	0.547	6.898 E-05	0.549	0.000	0.000	0.000	-0.0427	8.837 E-04
	Period (secs)								
	0.201			0.200			0.116		
	Frequency (Hz)								
	4.970			4.998			8.637		
	Modal damping ratio assumed in analyses								
	0.03			0.03			0.03		

TABLE 4.4 Dynamic characteristics of 8 - story superstructure (including 5% mass eccentricity). Modes higher than the 9th are not presented.

Floor	Mode								
	1			2			3		
	L Component	T Component	Rotational Component	L Component	T Component	Rotational Component	L Component	T Component	Rotational Component
8	0.000	0.285	3.440 E-05	0.286	0.000	0.000	0.000	-0.0213	4.598 E-04
7	0.000	0.268	3.237 E-05	0.269	0.000	0.000	0.000	-0.0200	4.326 E-04
6	0.000	0.235	2.842 E-05	0.236	0.000	0.000	0.000	-0.0176	3.798 E-04
5	0.000	0.204	2.466 E-05	0.205	0.000	0.000	0.000	-0.0153	3.296 E-04
4	0.000	0.165	1.993 E-05	0.166	0.000	0.000	0.000	-0.0124	2.664 E-04
3	0.000	0.119	1.442 E-05	0.120	0.000	0.000	0.000	-0.0089	1.927 E-04
2	0.000	0.082	0.986 E-05	0.082	0.000	0.000	0.000	-0.0061	1.317 E-04
1	0.000	0.041	0.500 E-05	0.042	0.000	0.000	0.000	-0.0031	0.668 E-04
	Period (secs)								
	1.147			1.140			0.651		
	Frequency (Hz)								
	0.872			0.877			1.537		
	Modal damping ratio assumed in analyses								
	0.03			0.03			0.03		

TABLE 4.4 Continued.

Floor	Mode								
	4			5			6		
	L Component	T Component	Rotational Component	L Component	T Component	Rotational Component	L Component	T Component	Rotational Component
8	0.000	-0.290	-3.505 E-05	-0.291	0.000	0.000	0.000	0.236	2.850 E-05
7	0.000	-0.165	-1.989 E-05	-0.166	0.000	0.000	0.000	-0.023	-0.279 E-05
6	0.000	0.032	0.386 E-05	0.032	0.000	0.000	0.000	-0.257	-3.101 E-05
5	0.000	0.154	1.858 E-05	0.154	0.000	0.000	0.000	-0.225	-2.713 E-05
4	0.000	0.231	2.795 E-05	0.232	0.000	0.000	0.000	-0.028	-0.034 E-05
3	0.000	0.242	2.926 E-05	0.243	0.000	0.000	0.000	0.189	2.283 E-05
2	0.000	0.198	2.392 E-05	0.199	0.000	0.000	0.000	0.248	2.996 E-05
1	0.000	0.111	1.341 E-05	0.111	0.000	0.000	0.000	0.171	2.065 E-05
	Period (secs)								
	0.424			0.422			0.266		
	Frequency (Hz)								
	2.357			2.371			3.756		
	Modal damping ratio assumed in analyses								
	0.03			0.03			0.03		

TABLE 4.4 Continued.

Floor	Mode								
	7			8			9		
	L Component	T Component	Rotational Component	L Component	T Component	Rotational Component	L Component	T Component	Rotational Component
8	0.237	0.000	0.000	0.000	0.022	-4.684 E-04	0.000	-0.223	-2.692 E-05
7	-0.023	0.000	0.000	0.000	0.012	-2.659 E-04	0.000	0.254	3.063 E-05
6	-0.258	0.000	0.000	0.000	-0.002	-0.516 E-04	0.000	0.188	2.270 E-05
5	-0.225	0.000	0.000	0.000	-0.012	2.483 E-04	0.000	-0.124	-1.493 E-05
4	-0.028	0.000	0.000	0.000	-0.017	3.735 E-04	0.000	-0.259	-3.129 E-05
3	0.189	0.000	0.000	0.000	-0.018	3.911 E-04	0.000	-0.025	-0.305 E-05
2	0.249	0.000	0.000	0.000	-0.015	3.197 E-04	0.000	0.177	2.138 E-05
1	0.171	0.000	0.000	0.000	-0.008	1.792 E-04	0.000	0.190	2.297 E-05
	Period (secs)								
	0.265			0.241			0.191		
	Frequency (Hz)								
	3.778			4.154			5.241		
	Modal damping ratio assumed in analyses								
	0.03			0.03			0.03		

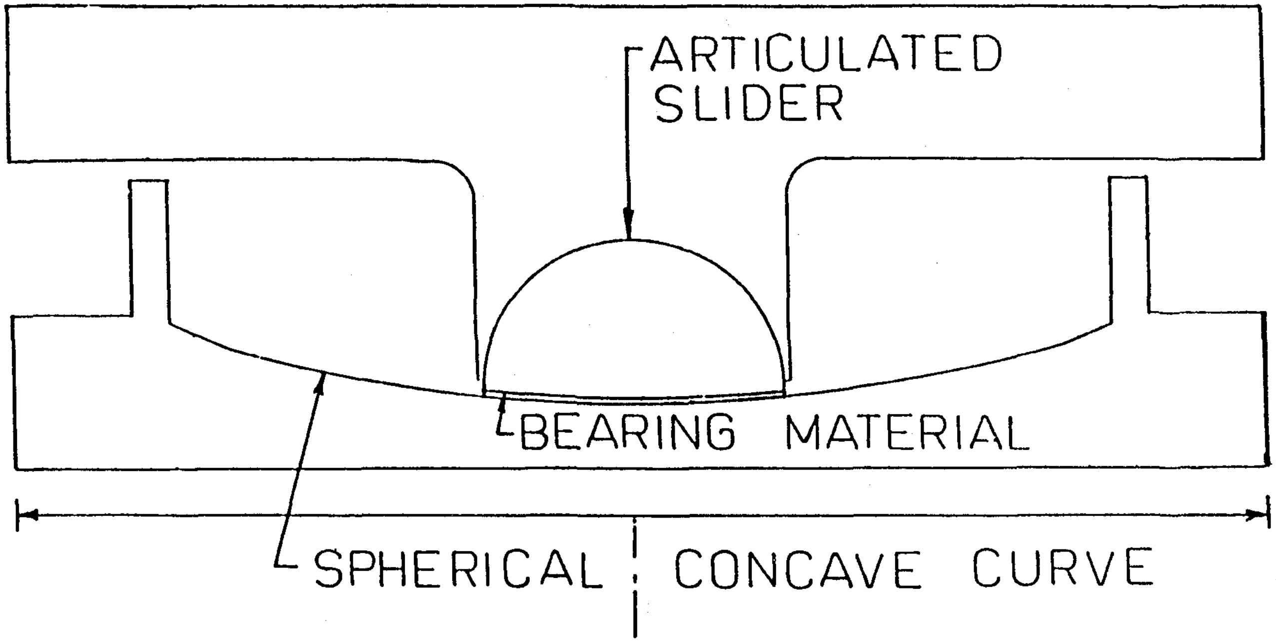


FIGURE 4-1 Typical section of a FPS bearing.

friction on velocity is a characteristic of Teflon based materials as described by Mokha et al, 1988 and 1990a. The FPS bearing acts like a fuse which is activated only when the earthquake forces overcome the minimum value of friction. When set in motion, the bearing develops a lateral force equal to the combination of the mobilized frictional force and the restoring force which develops as a result of the induced rising of the structure along the spherical surface. This restoring force is proportional to the displacement and the weight carried by the bearing and is inversely proportional to the radius of curvature of the spherical surface. Accordingly, the system has the following important properties:

(1)Rigidity for forces up to 0.05 times the weight.

(2)Lateral force which is proportional to the weight carried by the bearing. As a result of this significant property, the lateral force develops at the center of the mass, thus eliminating eccentricities at the isolation system level.

(3)Period of vibration in the sliding mode which is independent of the mass of the structure and related only to the radius of curvature of the spherical surface.

In addition to these properties, the Friction Pendulum System has other properties common to sliding isolation systems, such as low sensitivity to the frequency content of excitation and high degree of stability (Mokha et al 1988, Constantinou et al 1990b).

The lateral force that develops at an FPS bearing follows with excellent accuracy the following relationship:

$$F_b = \left(\frac{W}{R}\right) U_b + \mu(\dot{U}_b) W \, sgn(\dot{U}_b) \qquad (4.1)$$

in which W is the weight carried by the bearing, R is the radius of curvature of the bearing, μ is the coefficient of friction mobilized during sliding and U_b is the bearing displacement. The first term in equation 4.1 corresponds to the stabilizing tendency of pendulum action of the

FPS bearing with the quantity W/R representing the slope of the force-displacement relationship (see also Fig. 2-2). Accordingly, the period of vibration of the structure in its rigid body condition and with friction neglected is:

$$T_b = 2\pi\left(\frac{R}{g}\right)^{1/2} \tag{4.2}$$

From experimental measurements, it was found that the coefficient of friction follows the relation below, which was proposed by Constantinou et al, 1990b:

$$\mu(\dot{U}_b) = f_{\max} - Df\exp(-a\,|\dot{U}_b|) \tag{4.3}$$

in which f_{max} and (f_{max} - Df) are the maximum and minimum mobilized coefficients of friction respectively, and a is a parameter that controls the variation of the coefficient with the velocity of sliding.

It should be noted that T_b represents the period of free vibration of an isolated rigid structure. This is not the same as period T, equations 2.4 and 2.12 which is a ficticious quantity.

4.2.2 Isolation System Properties.

Each of the 45 isolators had identical properties of coefficient of friction and radius of curvature. The combinations used in this study are summarized in Table 4.5.

The axial load on each bearing was different due to the 5% mass eccentricity. Table 4.6 presents the portion of total weight W_T carried by each of the 45 bearings. For numbering of the bearings, refer to Figure 4-2. W_T equals 2560 kips for the 1 - story structure and 11520 kips for the 8 - story structure.

Each of the 45 FPS bearings was modeled by a bi-directional sliding element which conforms to the law of equation 4.3 and by a spring element of stiffness equal to W/R where W is the axial load carried by the bearing (see Table 4.6).

TABLE 4.5 Isolation System Properties.

Isolation System No#	Frictional properties			Geometrical Properties	
	fmax	fmin=fmax-Df	a (sec/in)	R (in)	T_b (sec)
1	0.10	0.05	0.9	39.132	2
2	0.05	0.025	0.9	88.048	3
3	0.10	0.05	0.9	88.048	3

TABLE 4.6 Axial load carried by each one of the bearings, for 1 - story and 8 - story isolated structure, as a proportion of the total weight W_T of the structure.

Bearing No#	Load/W_T
1	0.00625
2	0.0125
3	0.0125
4	0.0125
5	0.015625
6	0.01875
7	0.01875
8	0.01875
9	0.009375
10	0.0125
11	0.025
12	0.025
13	0.025
14	0.03125
15	0.0375
16	0.0375
17	0.0375
18	0.01875
19	0.0125
20	0.025
21	0.025
22	0.025
23	0.03125
24	0.0375
25	0.0375
26	0.0375
27	0.01875
28	0.0125
29	0.025
30	0.025
31	0.025
32	0.03125
33	0.0375
34	0.0375
35	0.0375
36	0.01875
37	0.00625
38	0.0125
39	0.0125
40	0.0125
41	0.015625
42	0.01875
43	0.01875
44	0.01875
45	0.009375

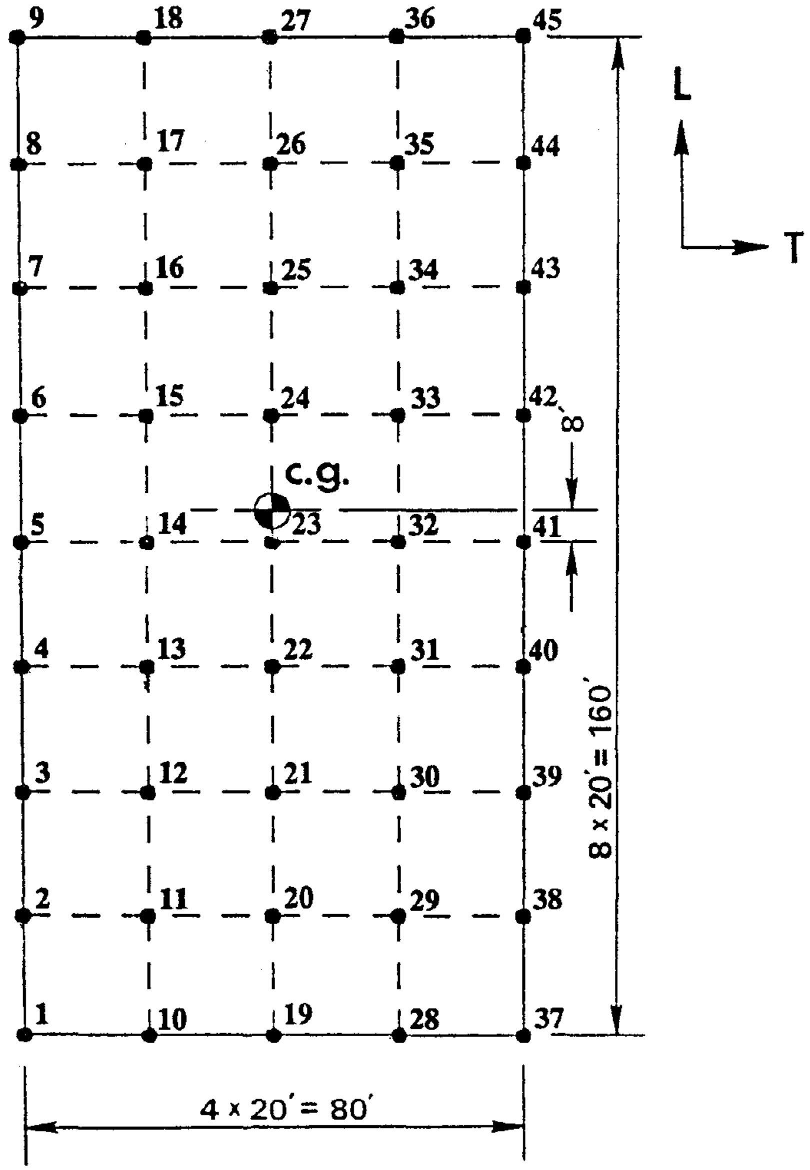


FIGURE 4-2 Plan view of the base of the building models used in the analyses and numbering of the bearings.

4.2.3 SEAOC Design Values for the Isolation System Properties

The intent of this study is to perform a series of nonlinear dynamic analyses and evaluate the response of isolated structures located in Seismic Zone 4 ($Z=0.4$), 15km or greater from an active fault ($N=1$), with various soil types (coefficient S) and supported by the Friction Pendulum System. These response values are compared with the design values that SEAOC specifies through a static analysis procedure. For this comparison, the SEAOC static analysis procedure was applied and the calculated isolation system displacements and base shear force values are listed in Table 4.7. The response quantities are presented as function of the soil type and isolation system properties (radius of curvature, R, and maximum coefficient of friction, f_{max}).

4.3 Program 3D-BASIS

The nonlinear analysis program 3D-BASIS (Nagarajaiah et al, 1989) was used in all the analyses made in this study. This program was developed as an efficient tool for analysis of base-isolated structures, in which the superstructure remains elastic during the earthquake and any nonlinear behavior is restricted to the isolation system. This program offers special options for the mathematical modeling of isolation systems, such as linear elastic, viscous, hysteretic and frictional elements with uni-directional and bi-directional behavior. All these elements are located at the base of the structure. The analysis methodology is based on the following assumptions:

(1) Superstructure remains elastic.

(2)Each floor has three degrees of freedom, X and Y translations and rotation about the center of mass of the floor.

(3)There exists a rigid slab at the base level that connects all isolation elements. The three degrees of freedom at the base are attached to the center of mass of the base.

TABLE 4.7 Displacement and base shear over weight ratio values according to SEAOC static analysis procedure for use in comparison with the results of nonlinear dynamic analyses performed in this study.

Sliding Isolation System Properties						Soil Type
R=39.132 in (T_b = 2 sec.) fmax=0.10		R=88.048 in (T_b = 3 sec.) fmax=0.05		R=88.048 in (T_b = 3 sec.) fmax=0.10		
D (in)	V_b/W	D (in)	V_b/W	D (in)	V_b/W	
2.809	0.172	5.277	0.110	3.113	0.135	S1 Rock/Stiff Soil Types Coefficient S = 1.0
5.717	0.246	10.189	0.166	6.320	0.172	S2 Medium Soil Sites Coefficient S =1.5
9.057	0.331	16.307	0.235	10.553	0.220	S3 Soft Soil Sites Coefficient S = 2.0

(4)Since three degrees of freedom per floor are required in the three-dimensional representation of the superstructure, the number of modes required for modal reduction is always a multiple of three. The minimum number of modes required is three.

(5)The isolation system is rigid in the vertical direction and torque resistance of individual isolation pads is neglected.

4.4 Comparison of Results Obtained by 3D-BASIS to Other Computer Programs.

Verifications were performed to compare the results from the 3D-BASIS program to a rigorous mathematical solution and to the DRAIN-2D program (Powell, 1973), to ensure the accuracy of the predicted results. For this reason, the response of the structural systems developed in Section 4.1 is evaluated for three different earthquake components by the three procedures. It should be noted that in order to comply to the limitations imposed by the program DRAIN-2D (2-Dimensional consideration only), the selected isolated buildings were subjected to only one horizontal earthquake component. By applying this component in the longitudinal direction (L-Direction) only, no mass-eccentricity effects could be considered. Accordingly, the accidental eccentricity in the longitudinal (L) direction was set equal to zero (see Section 4.1). Therefore, the verification analyses were confined to only a 2-Dimensional consideration. A total of 18 analyses were performed. The motions used in the comparison study are scaled records of earthquakes, as explained in Section 7.1

4.4.1 Comparison with Rigorous Mathematical Solution

4.4.1.1 Presentation of the Analytical Method

The response of the structures is analyzed using a lumped mass model. The equations of motion of the superstructure are:

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=-[M]\{1\}(\ddot{U}_g+\ddot{U}_b) \tag{4.4}$$

in which {U} is the vector of floor displacements with respect to the base, U_b is the base displacement with respect to the ground, and U_g is the ground displacement. A dot denotes differentiation with respect to time. Furthermore, [M], [C], and [K] are the mass, damping and stiffness matrices of the superstructure, respectively.

The equation of dynamic equilibrium of the entire system in the horizontal direction is:

$$\sum_{i=1}^{N} m_i(\ddot{U}_i+\ddot{U}_b+\ddot{U}_g)+m_b(\ddot{U}_b+\ddot{U}_g)+F_b=0 \tag{4.5}$$

in which m_i, $i=1,\ldots,N$ are the floor masses, m_b is the base mass and F_b is the force mobilized at the isolation interface. This force is given by:

$$F_b=\left(\frac{W}{R}\right)U_b+\mu(\dot{U}_b)WZ \tag{4.6}$$

in which μ is described by equation 4.3, W is the weight of the structure and Z is a variable governed by the following differential equation (Constantinou et al, 1990b):

$$Y\dot{Z}+\gamma\,|\,\dot{U}_b\,|\,Z\,|\,Z\,|+\beta\dot{U}_bZ^2-\dot{U}_b=0 \tag{4.7}$$

in which Y is the "yield" displacement (0.01 inches) and $\beta + \gamma = 1$. Z replaces the signum function in equation 4.1 and is used to account for the conditions of separation and reattachment. Equations 4.4 to 4.7 are reduced to a system of first order differential equations and numerically integrated using an adaptive integration technique with truncation error control which is appropriate for stiff differential equations (Gear, 1971). This approach has been described by Mokha et al 1990b.

4.4.1.2 Comparison

As shown in Figures 4-3 to 4-8, there is virtually no difference in the response that was computed from the two methods. Program 3D-BASIS is capable of reproducing thoroughly and capturing every detail of the response of the base displacement of the models used, as it was predicted by the rigorous mathematical solution.

4.4.2 Comparison with the DRAIN-2D Program

Program DRAIN-2D represents a standard nonlinear dynamic analysis computer program used by many structural engineers. It has been extensively tested and verified. The behavior of the FPS bearings could not be accurately modeled by DRAIN-2D, however, it was compared to 3D-BASIS because of its wide acceptance.

In modeling the frictional behavior with program DRAIN-2D, the bilinear hysteretic element was used. In this respect, the velocity dependence of the coefficient of friction was not accounted for. In the bilinear hysteretic element, the yield force was selected to be equal to $f_{max}W$ and the yield displacement to be 0.01 inches. The post yielding stiffness was selected to be equal to W/R. All bearings were lumped into a single element. The extremely small yield displacement required an accordingly very small time step of integration. Stable solutions were achieved with a time step of 0.002 seconds.

Figures 4-3 to 4-8 compare the base displacement time histories calculated from the two programs. The results demonstrate that 3D-BASIS and DRAIN-2D yield the same or almost the same displacement time histories. Some small differences in the details of the time histories are attributed to the velocity dependence of the coefficient of friction which was not accounted for in the DRAIN-2D solution.

Based on the above observation it may be concluded that:

(1)The velocity-dependent behavior of Teflon-based sliders is of secondary importance

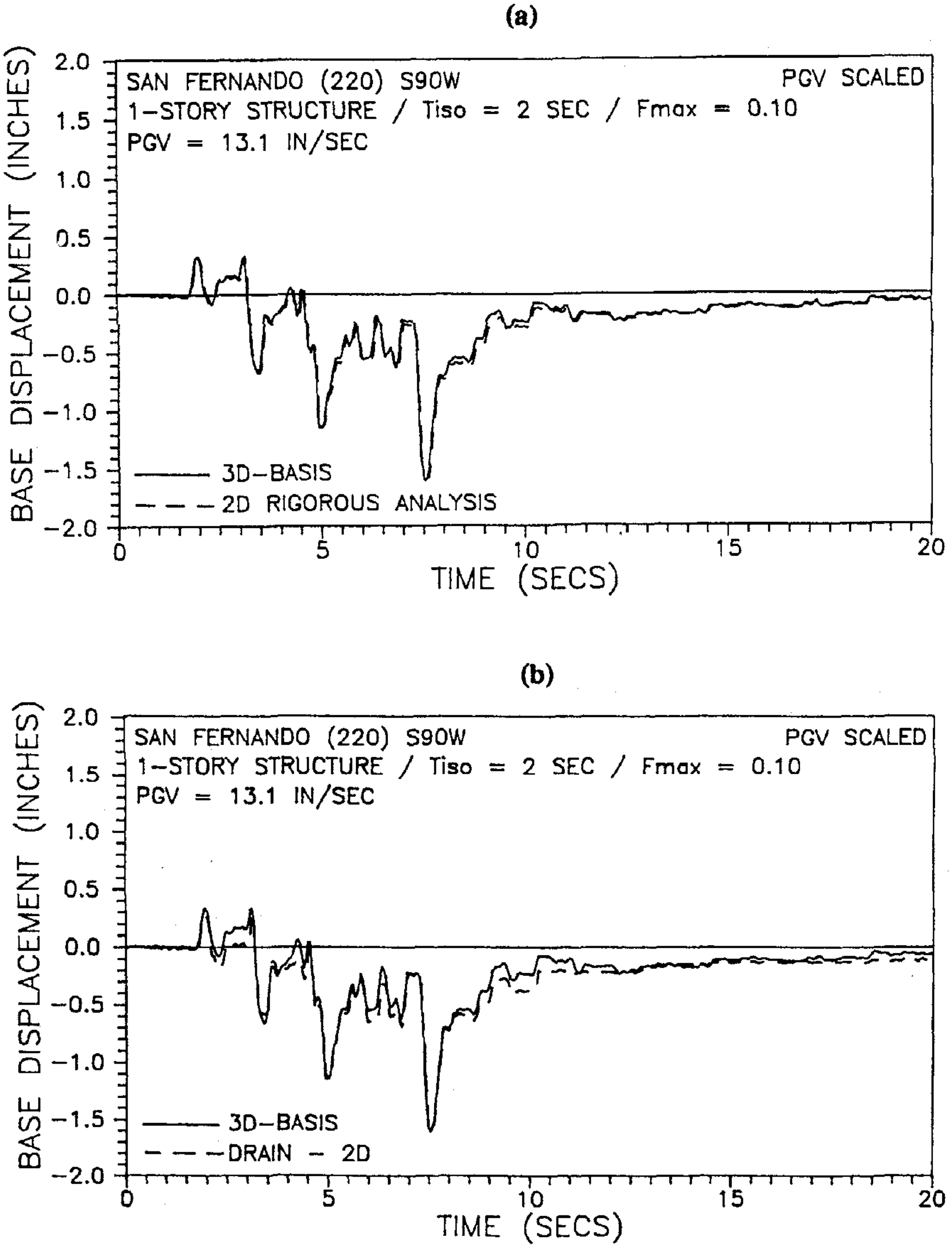


FIGURE 4-3 Comparisons of base displacement time histories obtained by programs 3D-BASIS, DRAIN-2D and a rigorous analysis program for 1 - story structure subjected to San Fernando (220) S90W earthquake component in the longitudinal direction. Scaling of the record is based on PGV according to Table 7.4

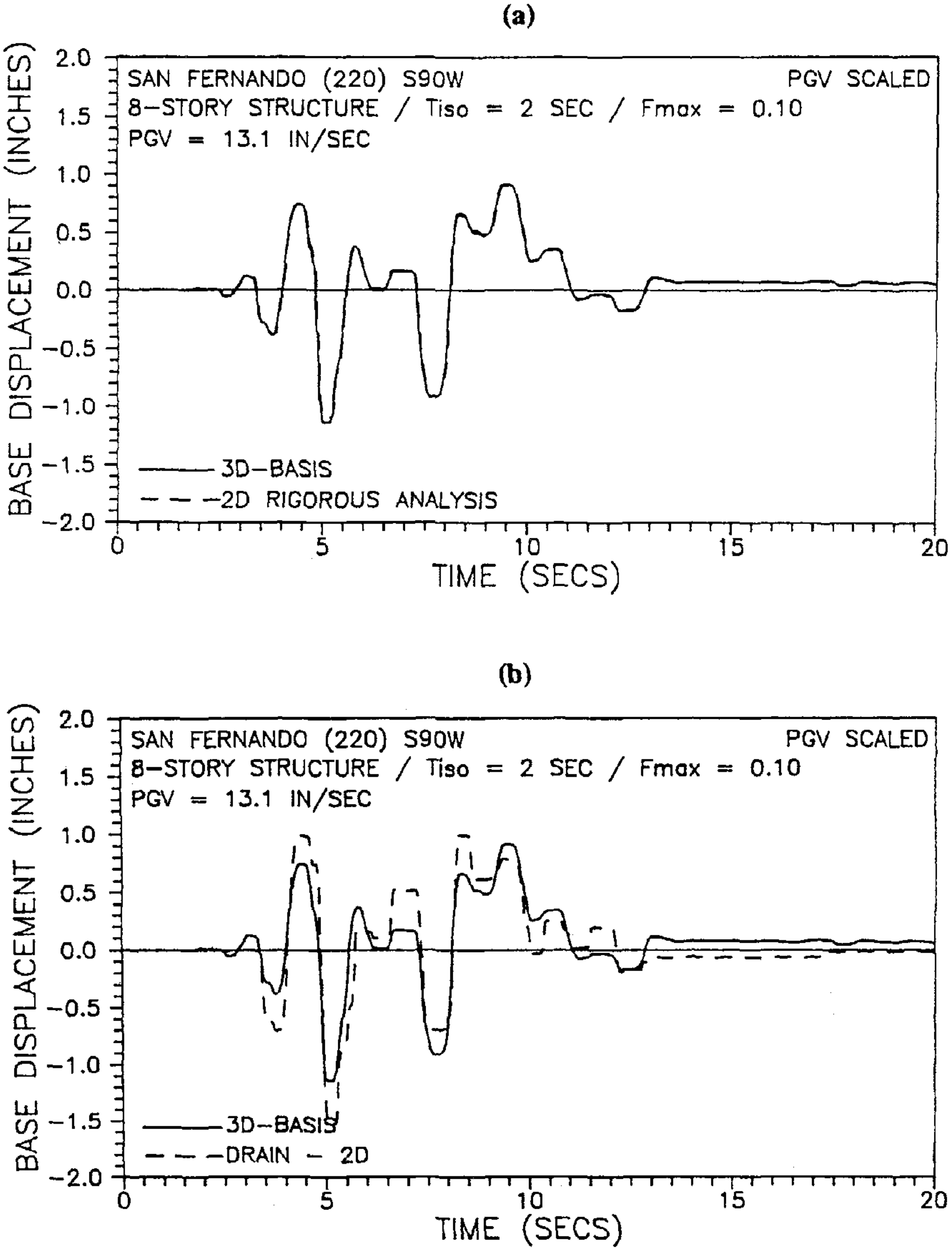


FIGURE 4-4 Comparisons of base displacement time histories obtained by programs 3D-BASIS, DRAIN-2D and a rigorous analysis program for 8 - story structure subjected to San Fernando (220) S90W earthquake component in the longitudinal direction. Scaling of the record is based on PGV according to Table 7.4

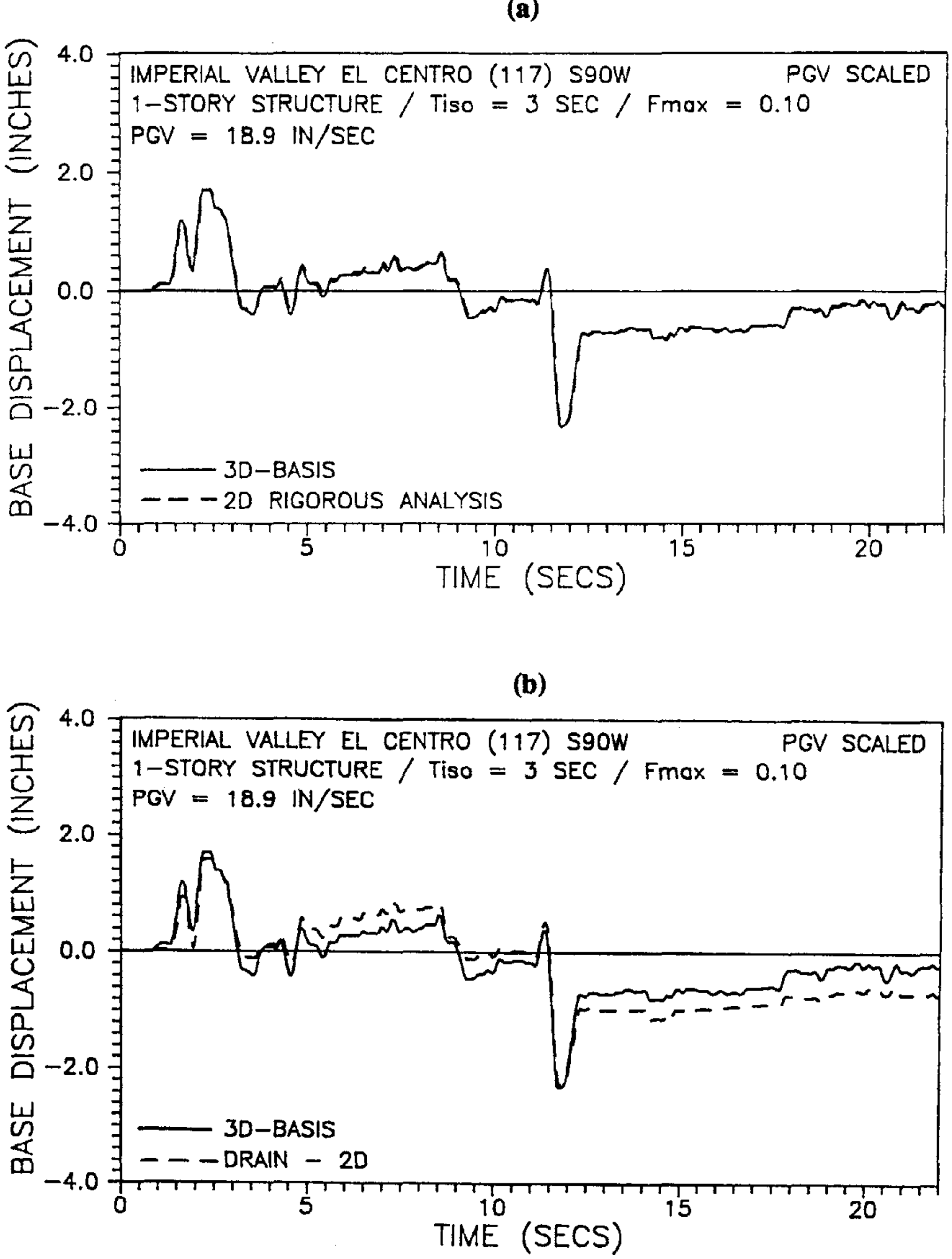


FIGURE 4-5 Comparisons of base displacement time histories obtained by programs 3D-BASIS, DRAIN-2D and a rigorous analysis program for 1 - story structure subjected to Imperial Valley El Centro (117) S90W earthquake component in the longitudinal direction. Scaling of the record is based on PGV according to Table 7.4

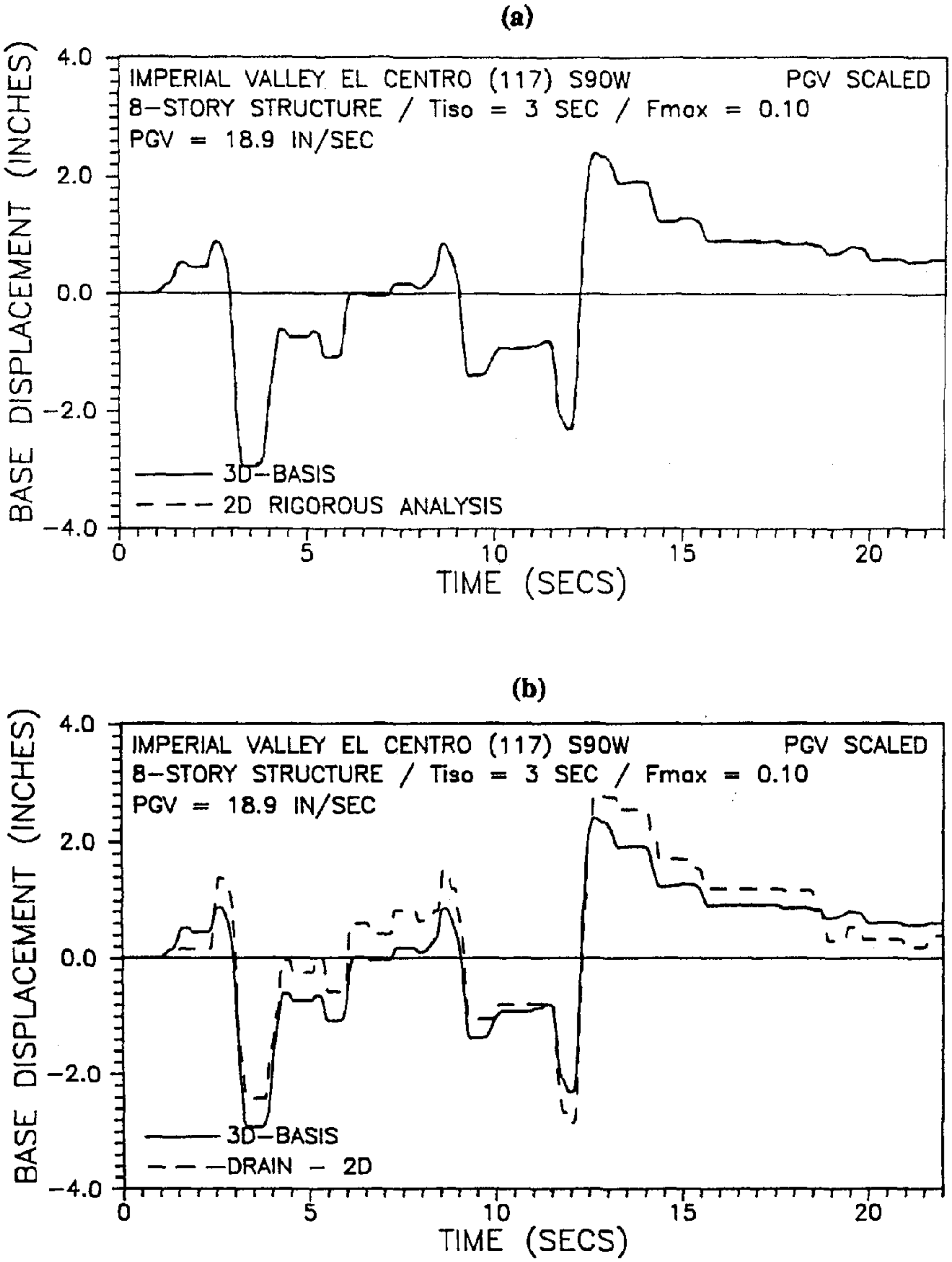


FIGURE 4-6 Comparisons of base displacement time histories obtained by programs 3D-BASIS, DRAIN-2D and a rigorous analysis program for 8 - story structure subjected to Imperial Valley El Centro (117) S90W earthquake component in the longitudinal direction. Scaling of the record is based on PGV according to Table 7.4

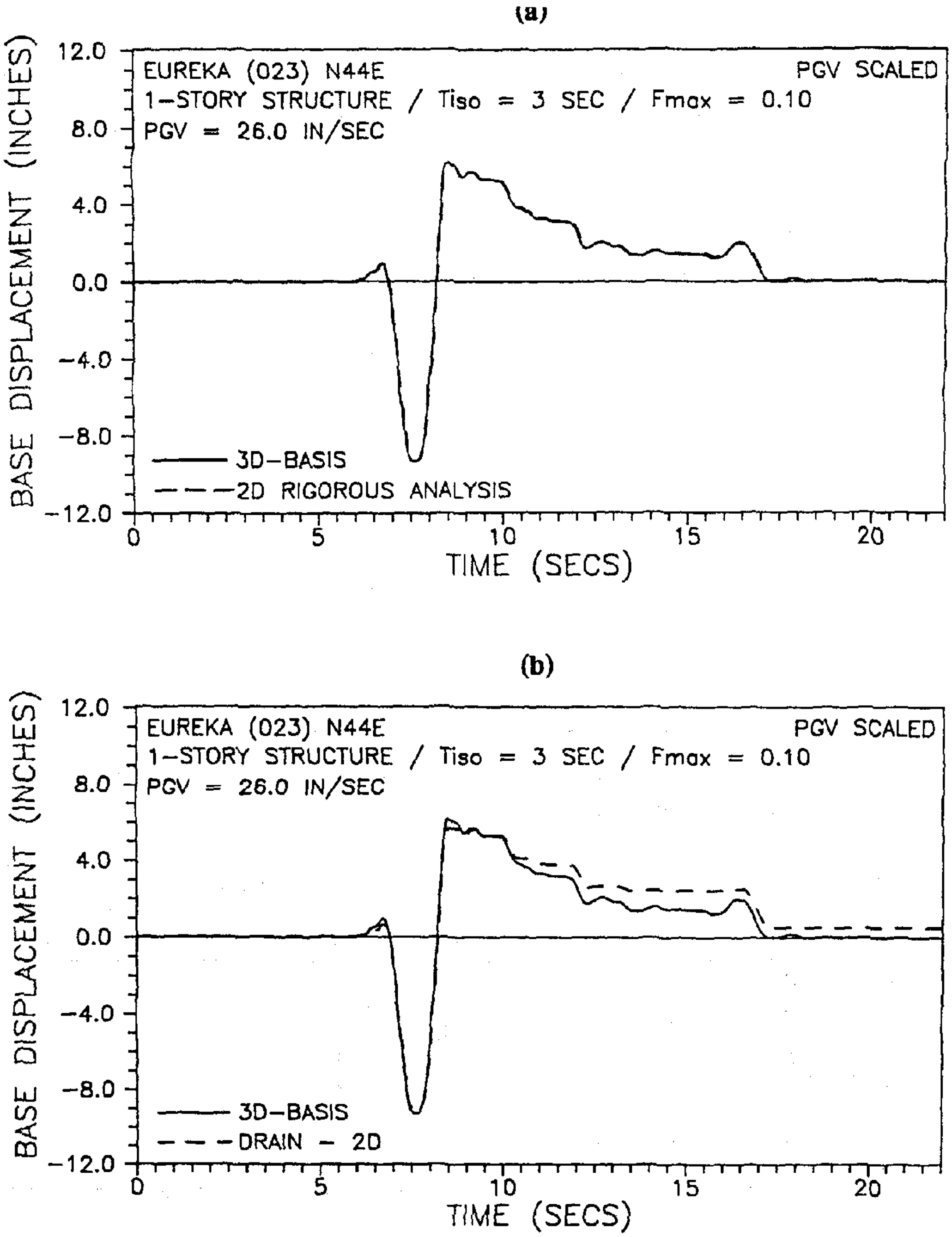


FIGURE 4-7 Comparisons of base displacement time histories obtained by programs 3D-BASIS, DRAIN-2D and a rigorous analysis program for 1 - story structure subjected to Eureka (023) N44E earthquake component in the longitudinal direction. Scaling of the record is based on PGV according to Table 7.4

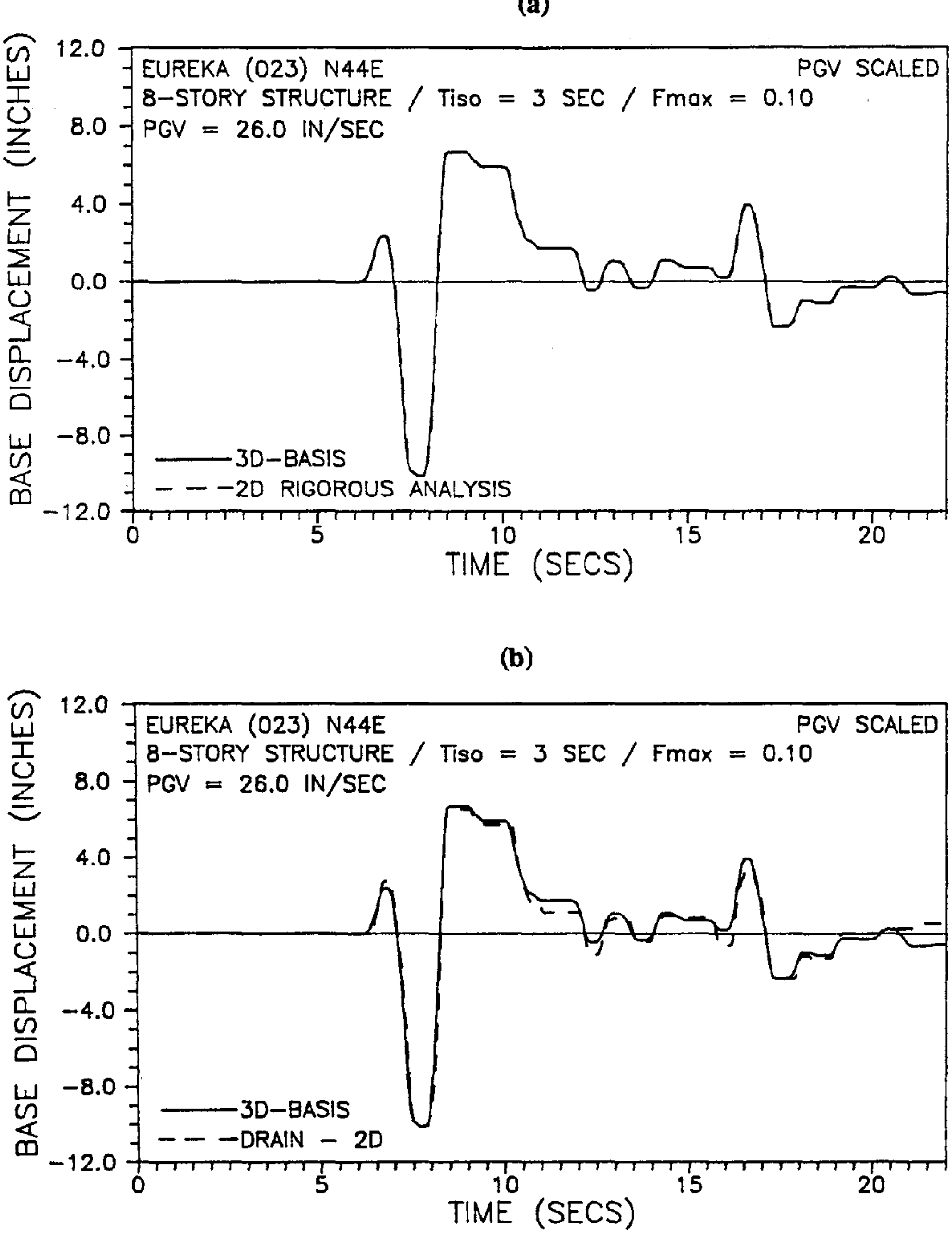


FIGURE 4-8 Comparisons of base displacement time histories obtained by programs 3D-BASIS, DRAIN-2D and a rigorous analysis program for 8 - story structure subjected to Eureka (023) N44E earthquake component in the longitudinal direction. Scaling of the record is based on PGV according to Table 7.4

to the calculation of peak isolator displacements, and

(2) Bilinear (non-velocity dependent) elements can be used to accurately calculate the displacement response of sliding isolation systems.

SECTION 5

COMPARISON OF SEAOC STATIC PROCEDURE TO SHAKE TABLE TESTS

Experimental data are essential for the verification of simplified design procedures like the SEAOC static procedure. In this regard, the experimental results from shake table testing of sliding isolation systems are utilized (Mokha et al, 1990b and 1991). Similar attempts for elastomeric and combined elastomeric - sliding systems have been reported by Chalhoub and Kelly, 1990 and Griffith et al, 1988.

5.1 Experimental Setup

5.1.1 Superstructure

The main purpose of the shake table tests carried by Mokha et al 1990b, was to investigate the feasibility of the Friction Pendulum System in isolating taller buildings with a large aspect ratio. Shake table tests were performed on a 1/4-scale artificial mass simulation model of a six-story steel moment resisting frame. In this model, the ratio of height to maximum distance between bearings was 2.25. The three bay model (Figure 5-1) had a weight of 51.4 kips. The fundamental frequency of the scaled model was 2.34 Hz or 1.17 Hz in prototype scale. This value is consistent with the behavior of a typical 6 - story moment resisting frame. The columns of the model were bolted to two heavy W14X90 sections and four bearings were placed between these beams and the shake table.

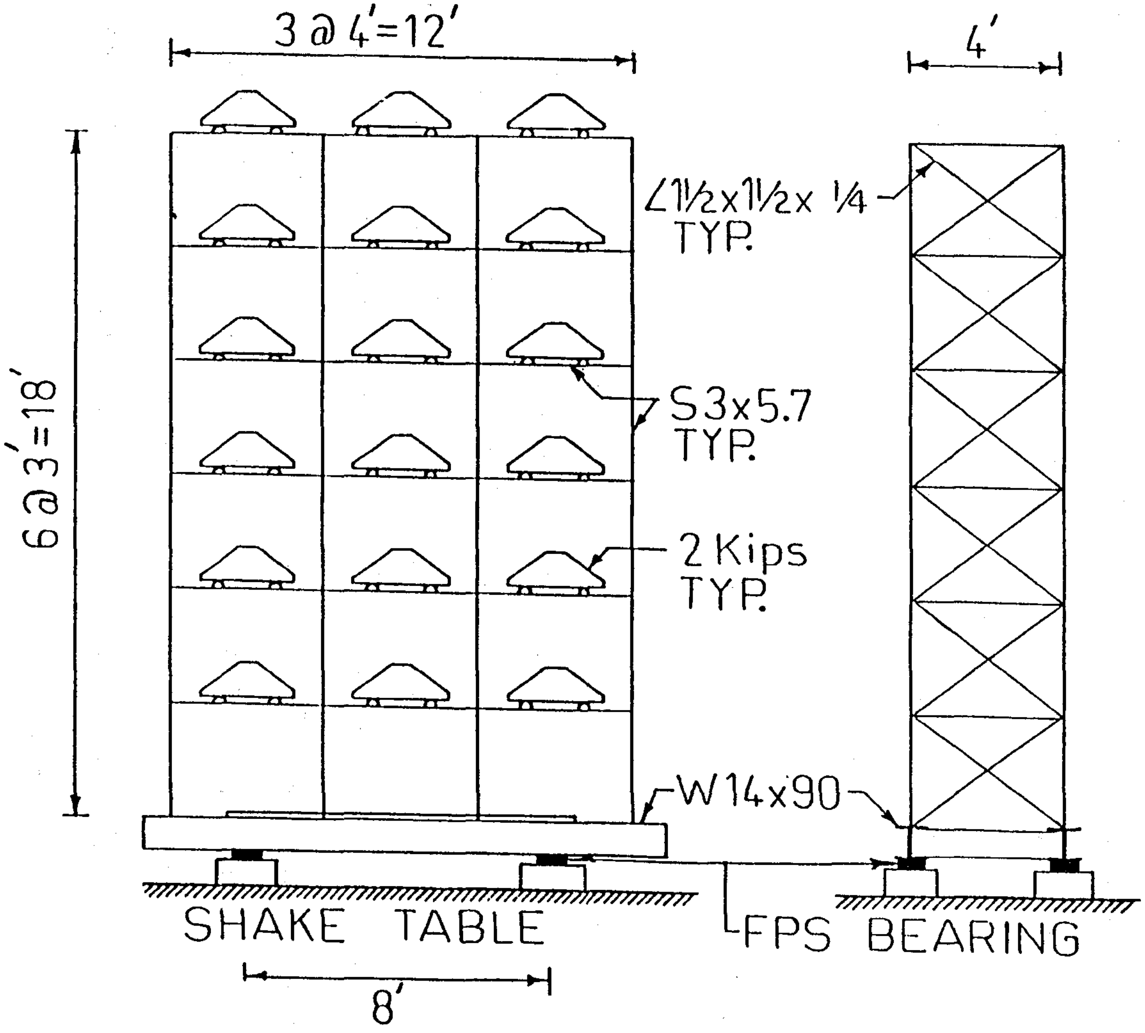


FIGURE 5-1 Model used for shake table testing .

5.1.2 Isolation System

The isolation system consisted of four FPS bearings which were placed under the base of the model at 8 feet distance as shown in Figure 5-1. In this configuration, the aspect ratio of the height of the model to distance between bearings is 2.25. The radius of curvature R, was equal to 9.75 inches (39 in. in prototype scale). This radius resulted in a period of 1 seconds (2 seconds in the prototype scale). Two different bearing materials were used:

(1)A form of woven Teflon under bearing pressure of about 20 ksi. The frictional properties of this material, when in contact with the polished metal surface, followed the law of equation 4.3 with $f_{max} = 0.075$, $Df = 0.035$ and $a = 1.1$ sec/inch.

(2)A material which carries the trade name Techmet B (product of Oiles Industry Co., Japan). Average pressure at the sliding interface was about 7 ksi. Under these conditions, this material exhibited a higher coefficient of friction than the other bearing material. The frictional properties of this material were $f_{max} = 0.095$, $Df = 0.045$ and $a = 0.9$ sec/inch.

5.1.3 Test Program

The isolated model was tested with six different earthquake motions. The characteristics of these earthquake motions are listed in Table 5.1. The records have significantly different frequency content, with Hachinohe and Mexico City being long period motions. The records were time scaled by a factor of two to satisfy similitude requirements of the quarter scale model. The time scaled Mexico City motion has a frequency content almost entirely at 1 Hz, which coincides with the rigid body mode frequency of the isolated model.

The earthquake tests were performed at varying peak acceleration levels for each of the signals. Each earthquake signal was run at increasing levels of peak table acceleration

TABLE 5.1 Earthquake records used in test program.

NOTATION	RECORD	PEAK ACCEL. (g)	PREDOMINANT FREQ. RANGE (Hz)	MAGNITUDE
El Centro S00E	Imperial Valley May 18, 1940 Component S00E	0.34	1 - 4	6.7
Taft N21E	Kern County July 21, 1952 Component N21E	0.16	0.5 - 5	7.2
Pacoima S74W	San Fernando February 9, 1971 Component S74W	1.08	0.25 - 2	6.4
Pacoima S16E	San Fernando February 9, 1971 Component S16E	1.17	0.25 - 6	6.4
Miyagi- Ken-Oki EW	Tohoku Univ. Sendai, Japan June 12, 1978 Component EW	0.16	0.5 - 5	7.4
Hachinohe NS	Tokachi-Oki Earthq., Japan May 16 1968 Component NS	0.23	0.25 - 1.5	7.9
Mexico City	SCT Building Seppt. 19, 1985 Component N90W	0.17	0.5	8.1

until the peak interstory drift reached approximately the value of 0.18 inches or 0.005 times the story height. This value has been analytically determined to be the limit of the elastic behavior of the structure.

5.2 SEAOC Static Analysis Procedure

As stated in Section 2, the design displacement formula prescribed by SEAOC is:

$$D = \frac{10ZNST}{B} \tag{5.1}$$

where T is the effective period of the system and B is a damping related term. Both depend on the isolation system properties and the displacement of the system. Parameters Z, N and S are dependent on the earthquake motion. For comparison of the predictions of equation 5.1 to the experimental results, parameters Z, N and S must be properly selected.

In the studies of Chalhoub and Kelly, 1990 and Griffith et al, 1988, parameter S was selected according to the frequency content of the motion. Product ZN was interpreted as the velocity related coefficient Av in accordance to ATC 3-06 (ATC, 1978).

The interpretation of product ZNS is different in this study. It is based on equation 5.1 and the 5% spectra of the earthquake motions. For 5% damping, parameter B = 1. Accordingly, the term 10ZNS is the ratio between D and T. Thus, in the displacement spectrum of an earthquake motion, this ratio is expressed as the tangent of a straight line starting from the origin of the axes and trying to approximate an ideal spectrum, where proportionality between the period (T) and the displacement (D) exists.

Accordingly, for the evaluation of the displacements of the model used in the shake table tests according to the SEAOC equivalent static method, an estimation of the factor ZNS for the respective earthquake excitations was preceded by applying the above mentioned concept. Figure 5-2 shows the 5% damping elastic displacement spectra of the earthquake motions (not scaled in time) that were used in the experiments and the proposed linear ones.

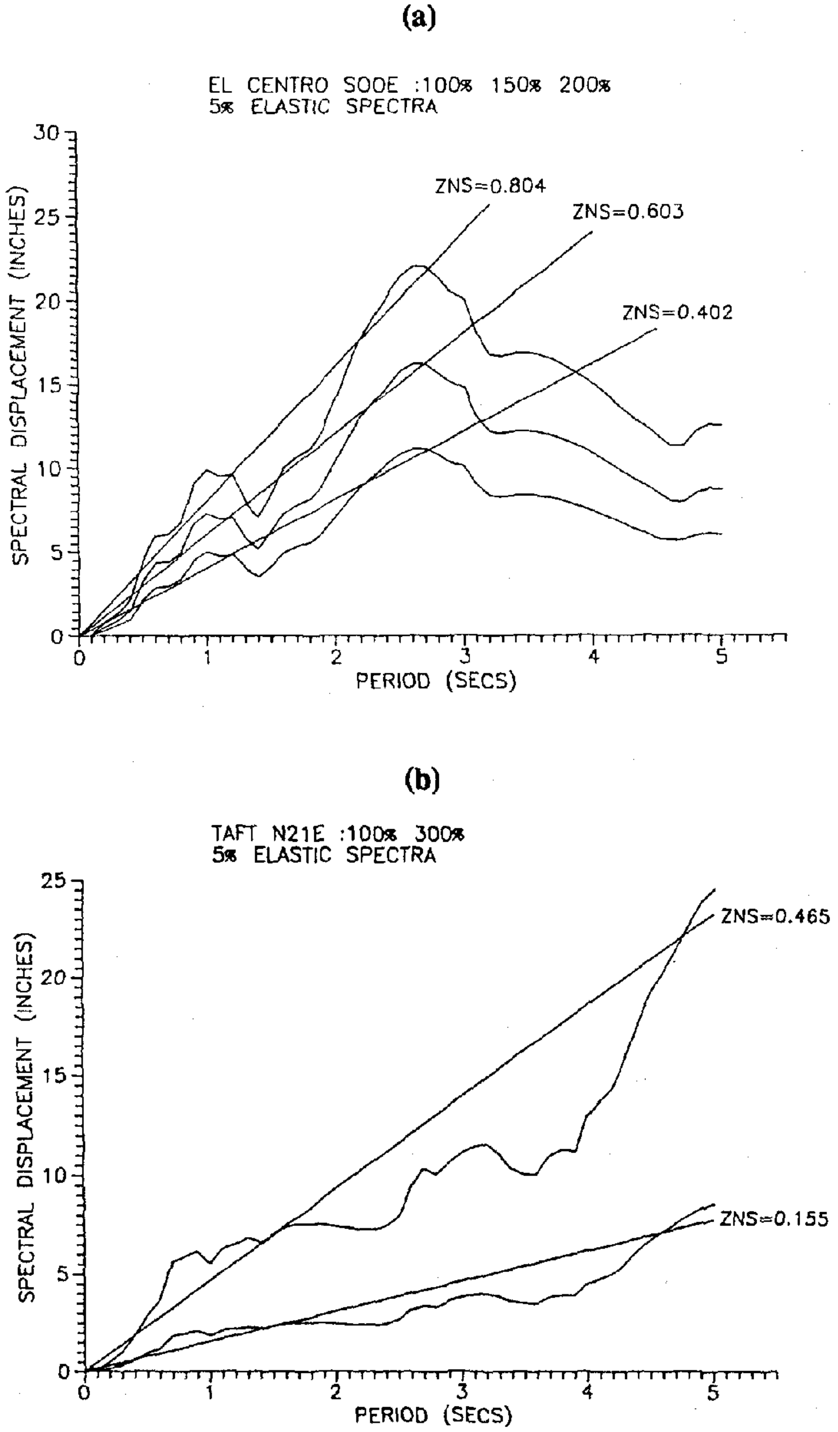


FIGURE 5-2 Displacement spectra for 5% Damping of records used in shake table testing (in prototype scale) and corresponding ZNS values.

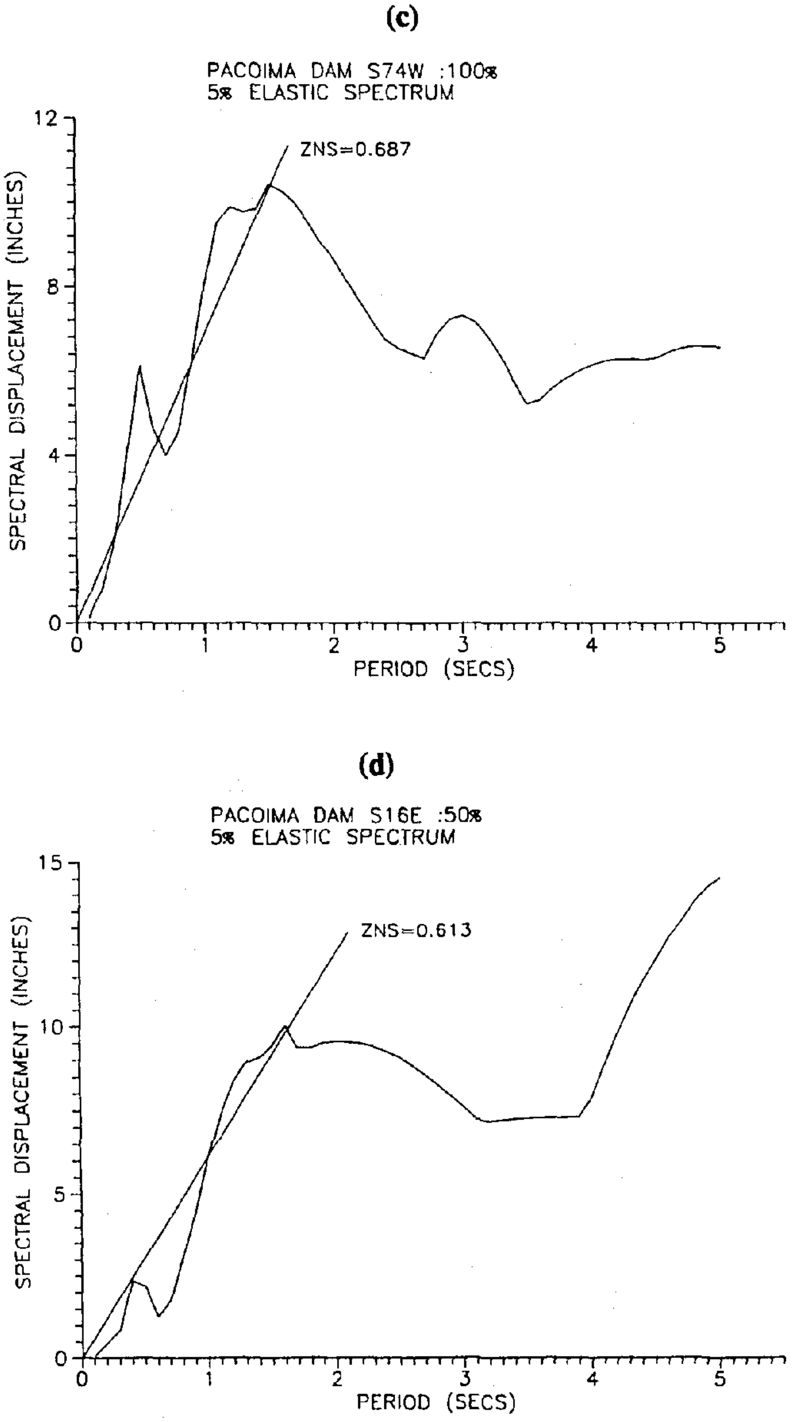
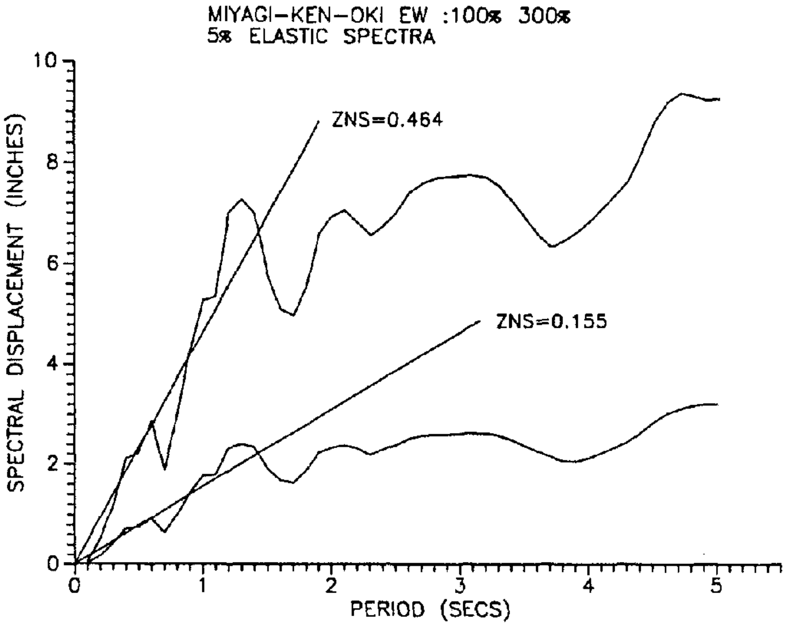


FIGURE 5-2 Continued.

(e)



(f)

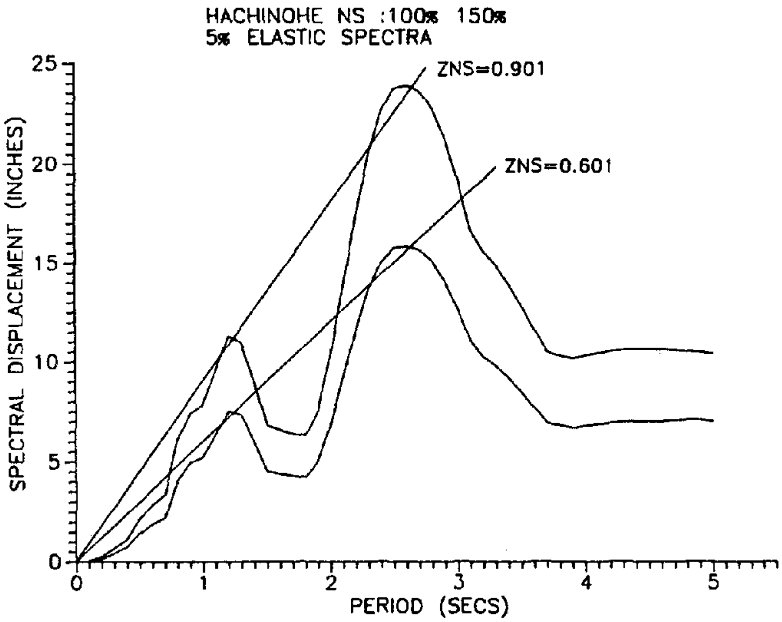


FIGURE 5-2 Continued.

(g)

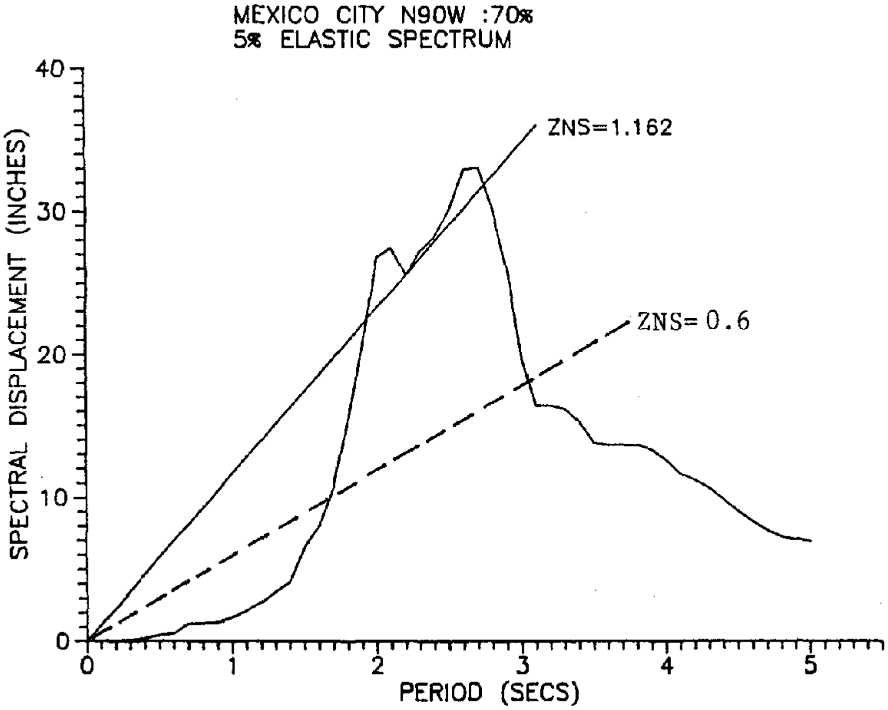


FIGURE 5-2 Continued.

The spectra were constructed from the recorded table motions. The percentage figure in Figure 5-2 represents the acceleration scaling of the original earthquake record. For example, the figure 200% implies an increase of the peak ground acceleration of the actual record by approximately a factor of 2.

The linear spectra were selected so that they give equivalent or conservative results when compared to the actual spectra in the period range from 1.0 to 2.0 seconds. This range contains the effective period of the tested sliding isloated structure. One should note, however, that the selection of the ZNS values is rather arbitrary and that several different values could fit the jagged shape of the test spectra at long periods. In some cases, the ZNS values in this study compare well with the ZNS values used by Chalhoub and Kelly, 1990 and Griffith et al, 1988. Table 5.2 compares ZNS values used in those studies and in this study.

The greatest uncertainty in the selected ZNS values occurs in the cases of long period motions like the Hachinohe and Mexico City motions. The spectra of these motions have a predominant peak which resembles the spectra of harmonic motions. An appropriate value of ZNS in these cases could be the one corresponding to a linear spectrum which matches the actual spectral displacement at the effective period of the isolation system.

5.3 Comparison of Experimental Results and Design Values According to SEAOC Static Analysis Procedure

Tables 5.3 and 5.4 provide information on the experimental results (extrapolated to prototype) of the displacement and the base shear coefficient of the tested model. The respective values according to SEAOC design formulae are also listed. For the calculation of the SEAOC design values, the procedure described in Section 2.3 was employed. The base shear over weight ratio was calculated without the 1.5 reduction factor to be consistent with the experimental value.

A direct observation can be made on the fact that SEAOC formulae consistently overestimate the displacement of the isolation system and the base shear coefficient, as they were recorded during the experiments. This observation is more intent in the case were the model was excited with long period motions. The ratio of SEAOC displacement to the experimental value for various earthquakes appears to be larger than those reported by Chalhoub and Kelly, 1990 and Griffith et al, 1988 for elastomeric and combined elastomeric/sliding isolation systems.

In the case of long period motions, like the Mexico City earthquake, the calculated SEAOC displacements are considerably larger than the experimental ones. Concentrating on the case of Mexico City 70% motion (Figure 5-2g), we repeat the calculations with a different interpretation of the ZNS value. We interpret this value as the one which results in a linear spectrum that intersects the actual displacement spectrum at the effective period of the isolation system. For the case of the system with $f_{max} = 0.075$ and $R = 39$ in., several iterations were needed before arriving at the modified ZNS value of 0.6, effective period $T = 1.67$ secs and displacement $D = 6.89$ in. The linear spectrum for $ZNS = 0.6$ is shown with dashed line in Figure 5-2g. The calculated displacement is considerably less than the one calculated with ZNS equal to 1.162 (Table 5.4). It is still, however, about 1.86 times the experimental one.

In the case of the base shear coefficient, SEAOC design values are also consistently conservative to the ones during the experiments. The ratio between the two values is lower than the ratio of the displacement values and this is attributed to the fact that the displacement and the base shear coefficient are not straight proportional, but they are rather related through equation 2.13, where the constant value of f_{max} mediates. It should be noted, however, that equation 2.13 (SEAOC formula for base shear in sliding isolation systems) predicts accurately the experimental results, provided that the experimental value of displacement is used. For example, if the experimental displacement of 4.92 in. for the El Centro 200% motion (see Table 5.3) is used in equation 2.13, the result is $V_b = 0.22W$ which is almost exact (0.218W).

TABLE 5.2 Comparison of the peak acceleration and ZNS values used by Chalhoub and Kelly, 1990 and Griffith et al, 1988 with the ones used in this study.

Motion	This study		Other studies	
	Peak Acceleration (g)	ZNS	Peak Acceleration (g)	ZNS
El Centro S00E	0.68	0.804	0.65	0.971
Pacoima Dam S16E	0.56	0.613	0.50	0.578
Taft N21E	0.53	0.465	0.74	0.825

TABLE 5.3 Shake table testing results (extrapolated to prototype) for the higher friction material ($f_{max} = 0.095$) and comparison to SEAOC design values.

Excitation	Peak Ground Acceleration (g)	ZNS	SEAOC design values		Experimental (extrapolated to prototype)		Ratio between SEAOC design values and experimental results	
			D (inch)	Vb/W *	D (inch)	Vb/W	D	Vb/W
El Centro SOOE 100%	0.34	0.402	3.01	0.172	1.58	0.126	1.91	1.37
El Centro SOOE 150%	0.51	0.603	6.00	0.249	3.05	0.157	1.97	1.59
El Centro SOOE 200%	0.68	0.804	9.46	0.338	4.92	0.218	1.92	1.55
Taft N21E 100%	0.17	0.155	0.63	0.111	0.45	0.101	1.40	1.10
Taft N21E 300%	0.53	0.465	3.92	0.196	3.57	0.173	1.10	1.13
Miyagiken Oki EW 100%	0.19	0.155	0.60	0.110	0.30	0.096	2.00	1.15
Miyagiken Oki EW 300%	0.57	0.464	3.85	0.194	2.10	0.138	1.83	1.41
Hachinohe NS 100%	0.22	0.601	5.89	0.246	2.27	0.152	2.59	1.62
Hachinohe NS 150%	0.36	0.900	11.40	0.387	4.48	0.199	2.54	1.94
Pacoima S74W 100%	0.92	0.687	7.25	0.281	5.60	0.203	1.29	1.38
Pacoima S16E 50%	0.57	0.613	6.12	0.252	4.44	0.198	1.38	1.27

* Without 1.5 reduction factor.

TABLE 5.4 Shake table testing results (extrapolated to prototype) for the lower friction material (f_{max} = 0.075) and comparison to SEAOC design values.

Excitation	Peak Ground Acceleration	ZNS	SEAOC design values		Experimental (extrapolated to prototype)		Ratio between SEAOC design values and experimental results	
	(g)		D (inch)	Vb/W *	D (inch)	Vb/W	D	Vb/W
El Centro SOOE 100%	0.34	0.402	3.56	0.167	1.73	0.114	2.06	1.46
El Centro SOOE 200%	0.68	0.804	10.94	0.356	7.04	0.243	1.55	1.47
Taft N21E 100%	0.17	0.155	0.79	0.095	0.54	0.090	1.46	1.06
Taft N21E 300%	0.55	0.465	4.48	0.190	4.16	0.173	1.08	1.10
Miyagiken Oki EW 100%	0.19	0.155	0.79	0.095	0.36	0.088	2.19	1.08
Miyagiken Oki EW 300%	0.56	0.464	4.48	0.190	2.24	0.123	2.00	1.54
Hachinohe NS 100%	0.22	0.601	6.74	0.248	2.35	0.126	2.87	1.97
Hachinohe NS 150%	0.35	0.901	12.84	0.404	5.44	0.201	2.36	2.01
Pacoima S74W 100%	0.92	0.687	8.50	0.293	6.08	0.198	1.40	1.48
Pacoima S16E 50%	0.56	0.613	7.00	0.254	5.12	0.195	1.37	1.30
Mexico N90W 40%	0.07	0.664	8.08	0.282	0.18	0.087	44.89	3.24
Mexico N90W 60%	0.11	0.996	15.06	0.461	1.05	0.116	14.34	3.97
Mexico N90W/ 70%	0.12	1.162	18.96	0.561	3.70	0.176	5.12	3.19

* Without 1.5 reduction factor.

In this respect, the SEAOC formula for the base shear in sliding isolation systems (equation 2.13) is exact provided that the design displacement, D, is accurately estimated. Accordingly, for the evaluation of SEAOC design procedure, we shall concentrate only on comparisons of the design displacement (equation 2.1) to the dynamic analysis results.

Concluding this section, we note that the SEAOC static procedure overpredicts unidirectional test displacements. The amount of overprediction is difficult to quantify because of the difficulty in selecting ZNS values to represent a single earthquake motion history.

SECTION 6

EVALUATION OF RESPONSE FOR ARTIFICIAL EARTHQUAKES COMPATIBLE TO DESIGN SPECTRA

An alternate approach for the evaluation of the response of sliding seismically isolated structures under earthquake excitations is examined in this section. Specifically, a series of analyses was made where the two building models discused in Section 4 were subjected to artificially generated earthquake motions. The intent of this methodology is to create appropriate simulated time histories, which are compatible with specified response spectra, and, using these simulated motions, to perform nonlinear dynamic analyses. With this concept in mind, a collection of response data was created from structures that are subjected to earthquake excitations, whose response spectra closely match the shape of the recommended spectra for use in building codes, according to the ATC 3-06.

The two building models were subjected to excitation only in their transverse (T) direction. This way, the effects of mass eccentricity were taken into full account, since in both models a 5% mass eccentricity existed only in the transverse direction. This assumption allowed investigation of the potential for rotation of the isolation system and the calculation of corner bearing displacements.

Recognizing that the above approach is limited to one - directional excitation, analyses were also performed for the case of the 8 - story structure with bi-directional simulated excitation. This excitation consisted of 100% of the simulated motion in the transverse (T) direction and a portion (83%) of the same simulated motion in the longitudinal (L) direction, acting simultaneously. The 100%-83% combination is consistent with the dynamic time history analysis approach of SEAOC as described in Section 2.2.2.3 ($(L^2+T^2)^{1/2} = (1+0.83^2)^{1/2} = 1.3$).

A total of 9 earthquake motions was generated. Grouped by three, their spectra sufficiently approximated the spectra prescribed by ATC 3-06 for soil types S1,S2 and S3. All of the records had a peak ground acceleration of 0.4g, which is the effective PGA for Seismic Zone 4. This zone, as referred to the introduction of this study, was the zone of interest for all the series of analyses that were made within this work, since it is both the zone of highest seismicity, and the zone where most isolated structures have been and will be constructed. The duration of the generated accelerograms was 20 seconds for those that corresponded to soil types S1 and S2, and 30 seconds for the ones that were created according to the S3 design spectrum. This selection was made with the assumption that those values of time intervals could be considered representative of the duration of main intensity intervals of real earthquakes, as recorded in soil types S1, S2 and S3.

The time histories of acceleration of the simulated motions and their 5% damping elastic spectra are presented in Figures 6-1 to 6-9. The target spectra are also included in these figures. It may be seen that the response spectra of the simulated motions closely match the target spectra over the entire range of periods of interest in this study. The simulation of the earthquake motions was based on the approach of Gasparini and Vanmarke, 1976.

6.1 Comparison of Time History Analysis Results for One-directional Excitation to SEAOC Design Formulae

In the analyses that were performed, the two building models discussed in Section 4 were combined with the three different isolation system models and excited by 9 artificial earthquake motions, grouped by three to represent site conditions S1, S2 and S3. A total of 54 analyses was performed.

Tables 6.1 and 6.2 summarize the results of the maximum displacements for the 1-story and the 8-story structure, respectively. Tables 6.3 to 6.8 present results of the analyses in more detail. It should be noted that for every combination of isolation system, superstructure and

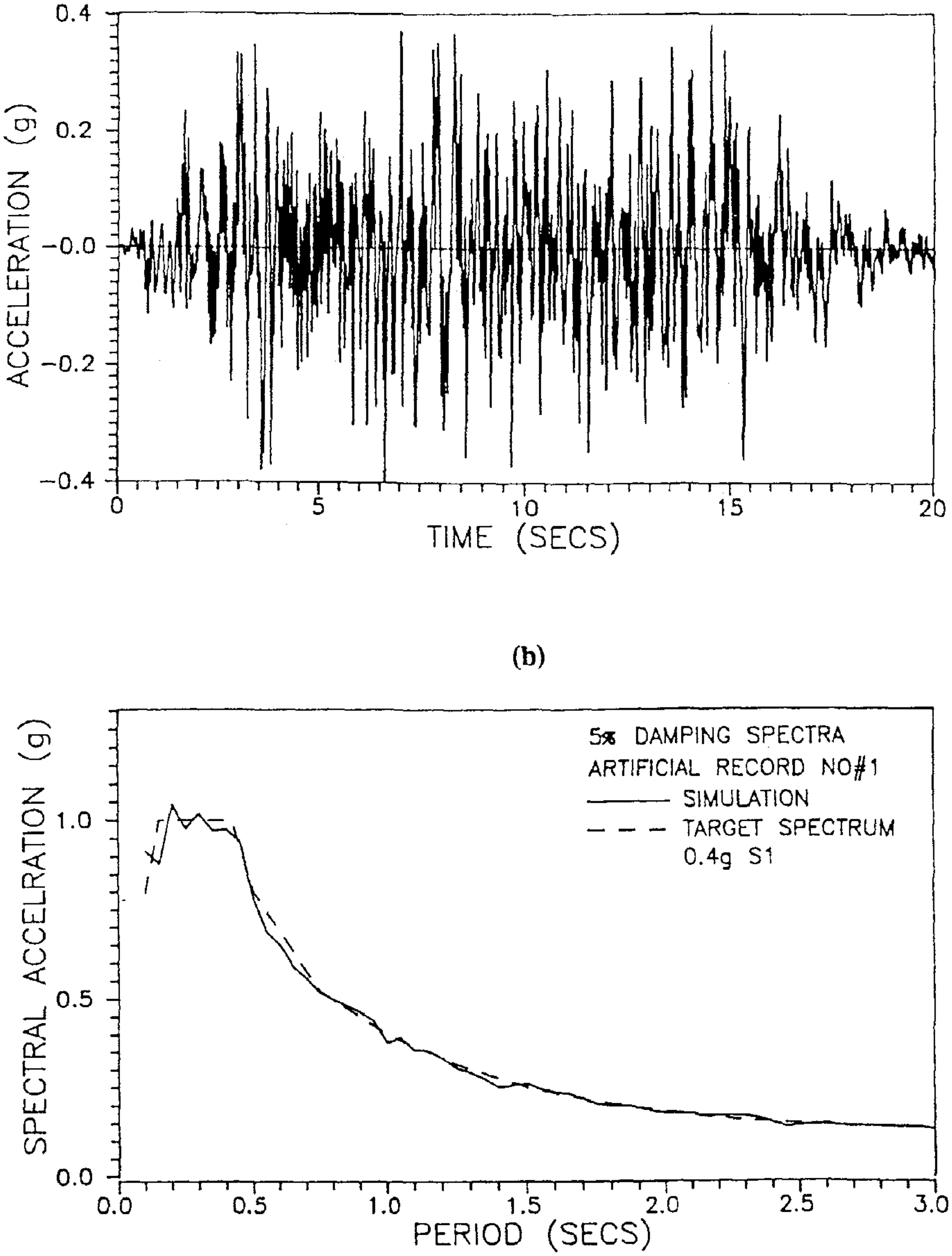


FIGURE 6-1 Artificial record No#1 compatible with 0.4g S1 Design Spectrum and comparison of its spectrum with the target spectrum.

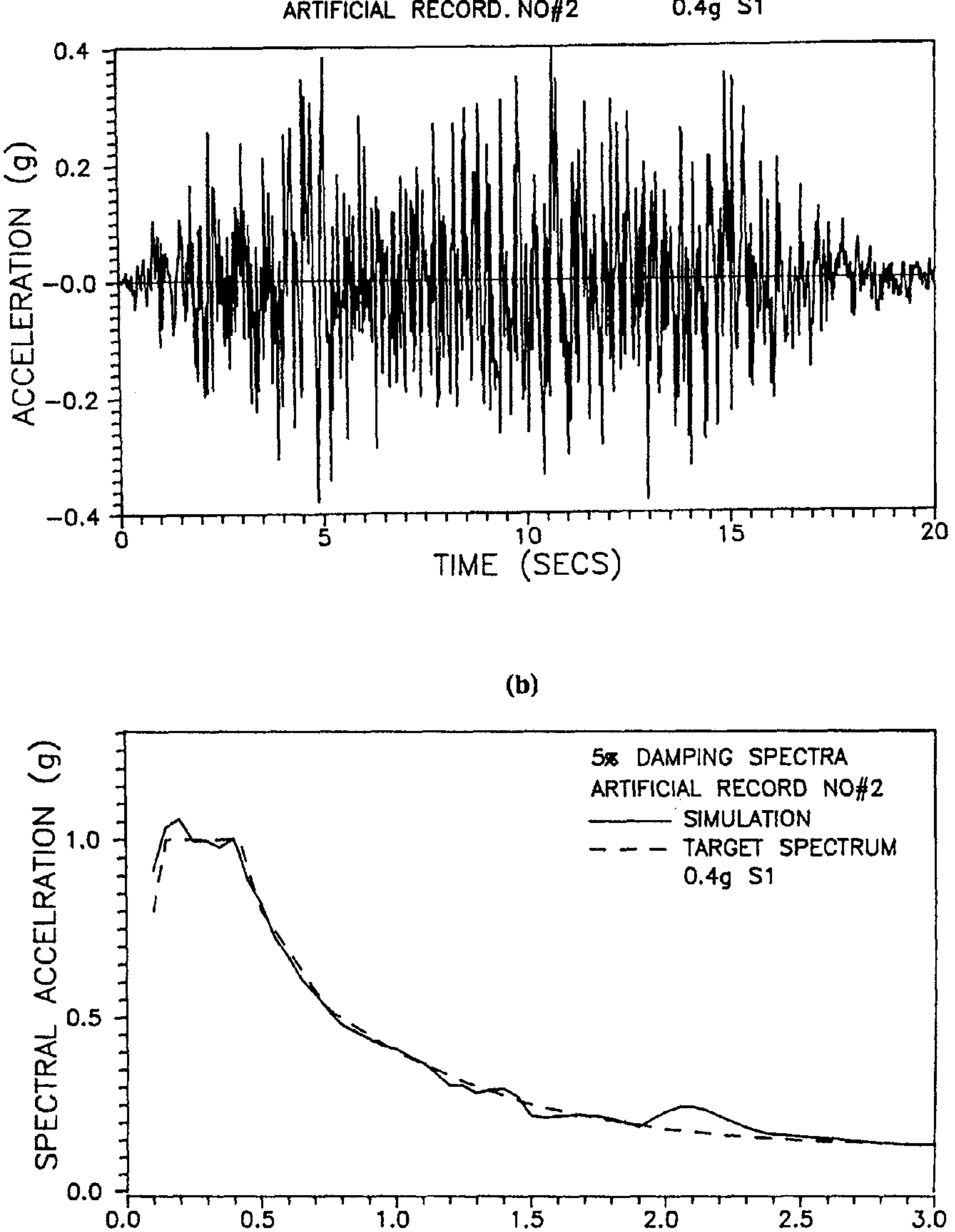
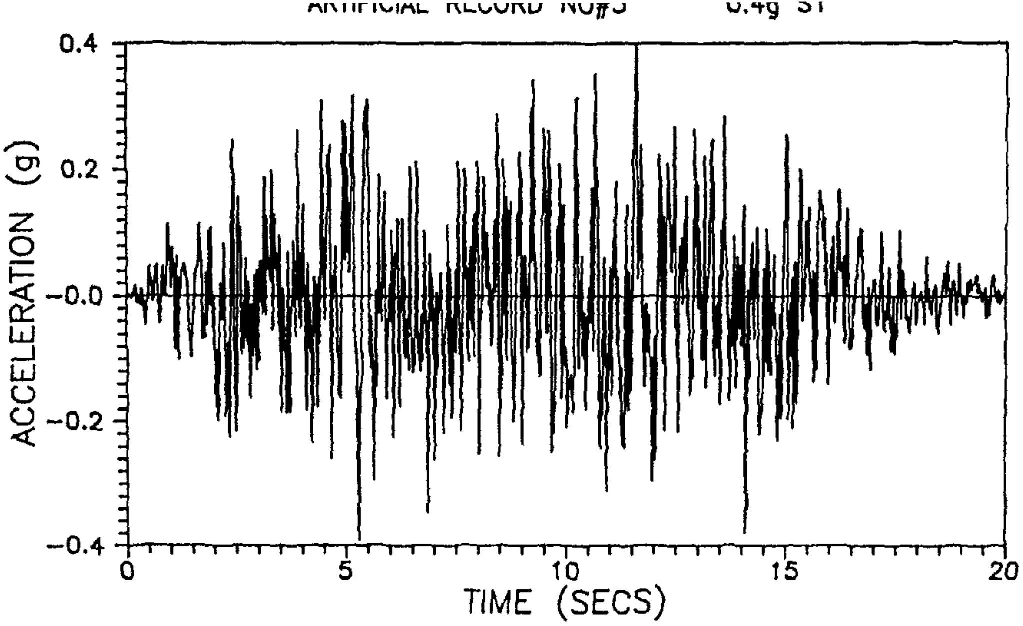


FIGURE 6-2 **Artificial record No#2 compatible with 0.4g S1 Design Spectrum and comparison of its spectrum with the target spectrum.**



(b)

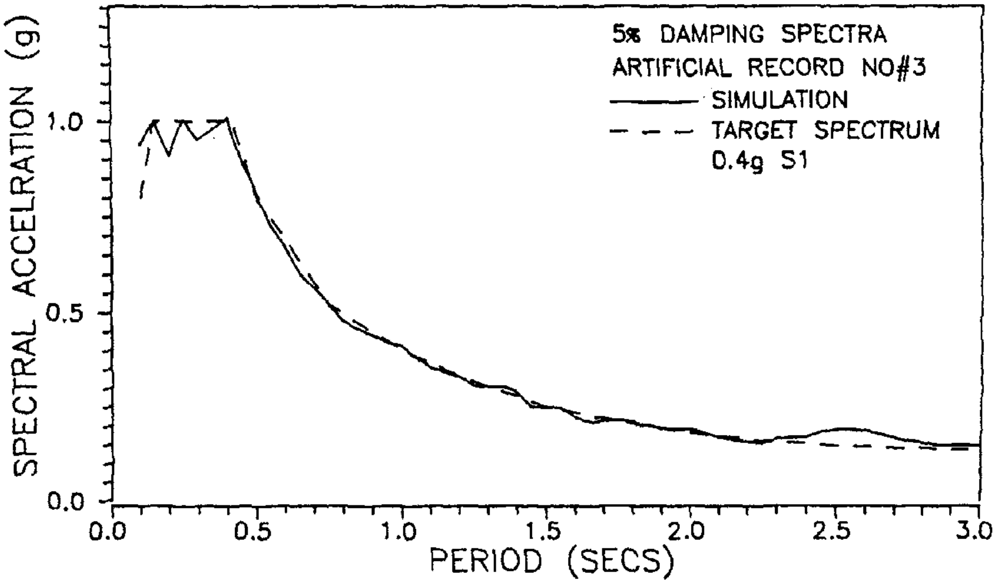


FIGURE 6-3 Artificial record No#3 compatible with 0.4g S1 Design Spectrum and comparison of its spectrum with the target spectrum.

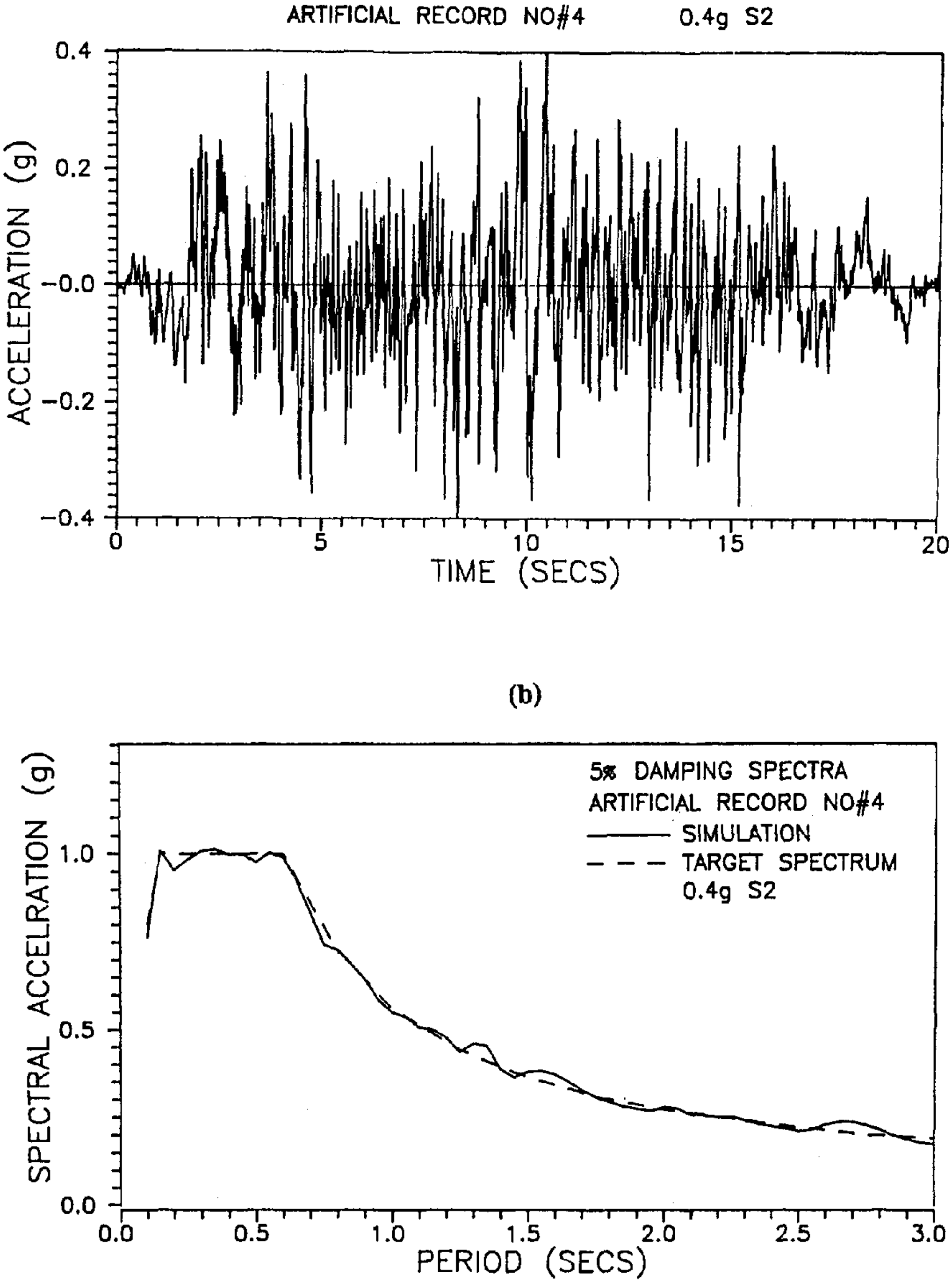


FIGURE 6-4 Artificial record No#4 compatible with 0.4g S2 Design Spectrum and comparison of its spectrum with the target spectrum.

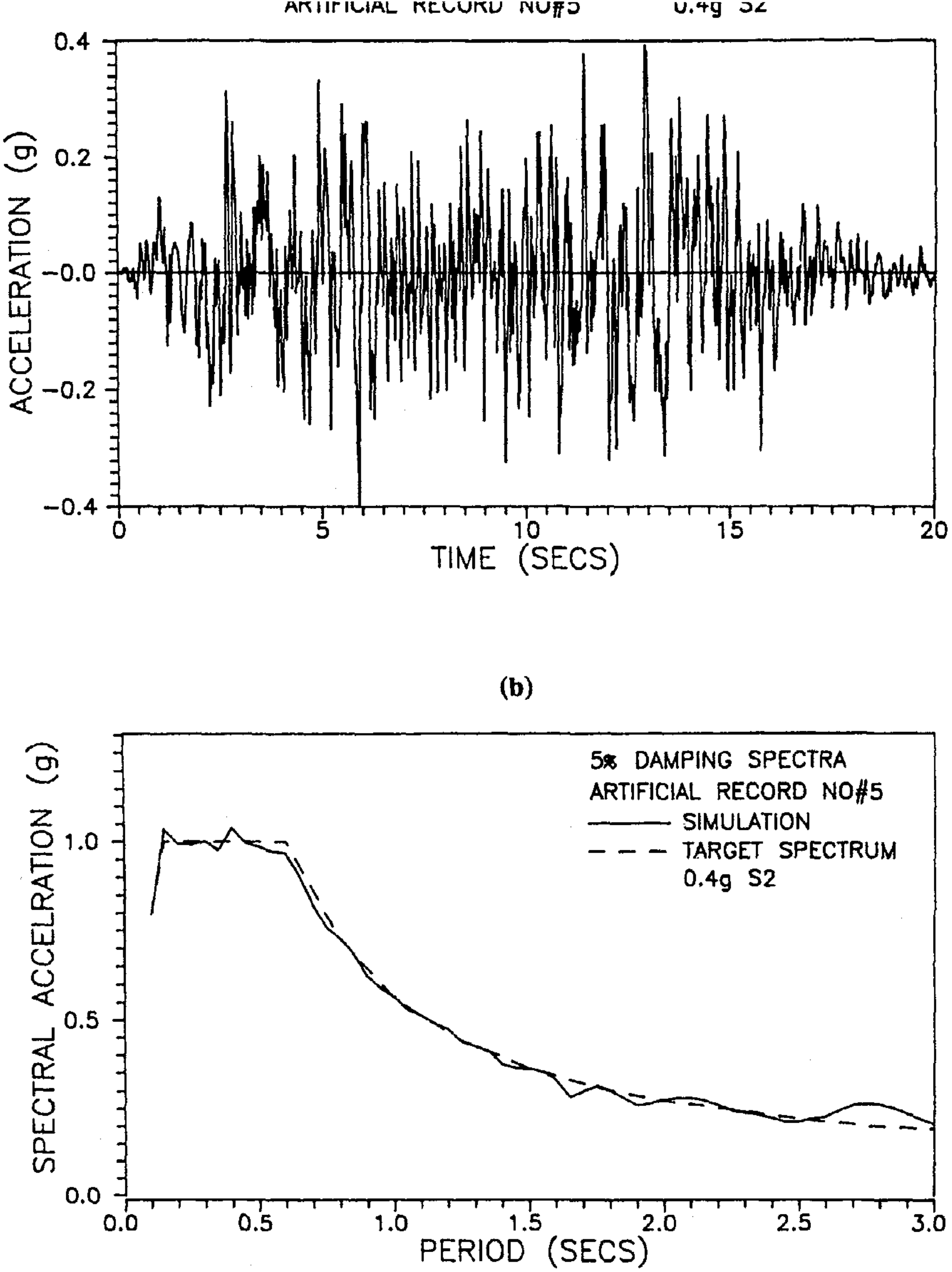


FIGURE 6-5 Artificial record No#5 compatible with 0.4g S2 Design Spectrum and comparison of its spectrum with the target spectrum.

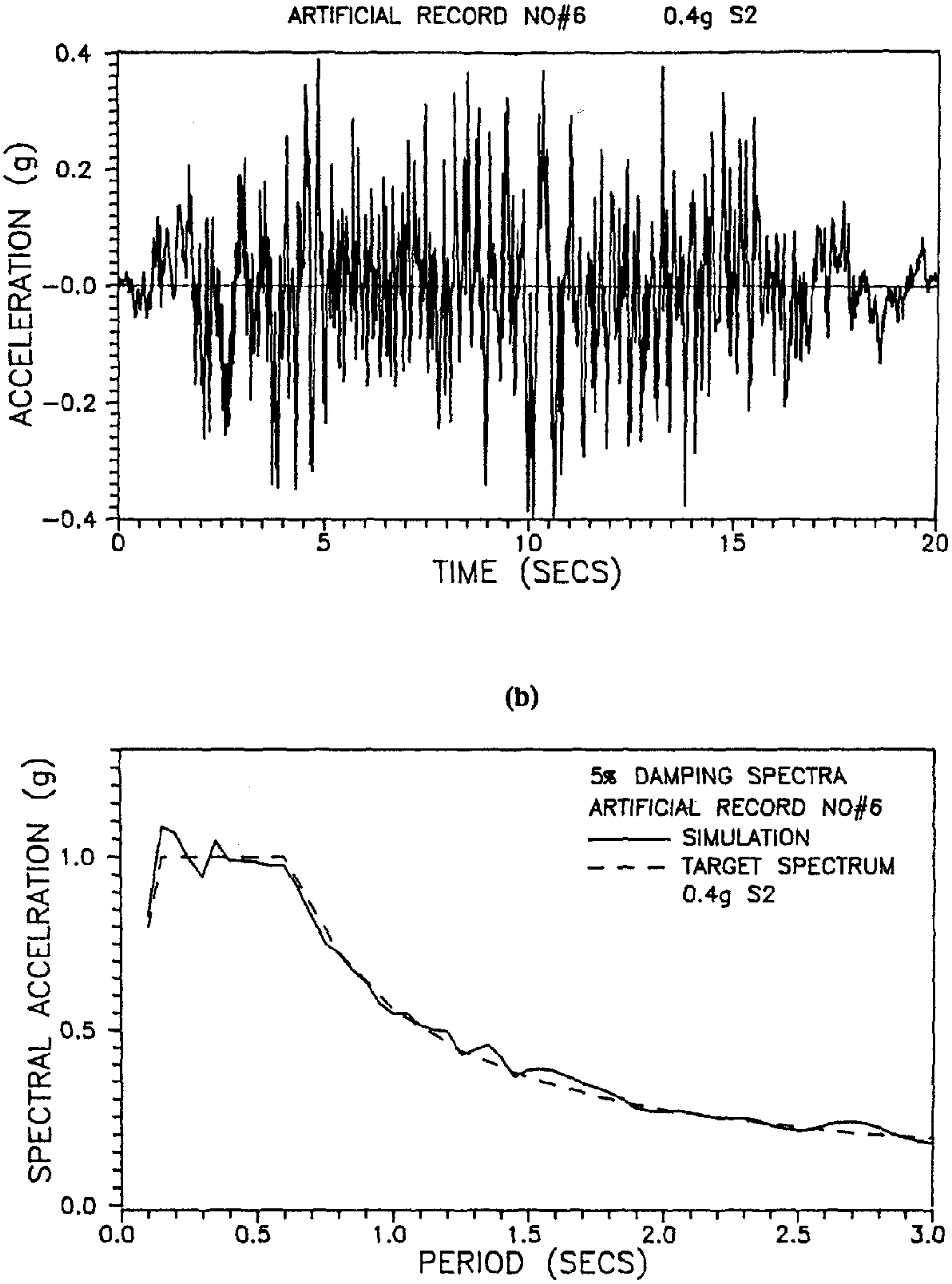
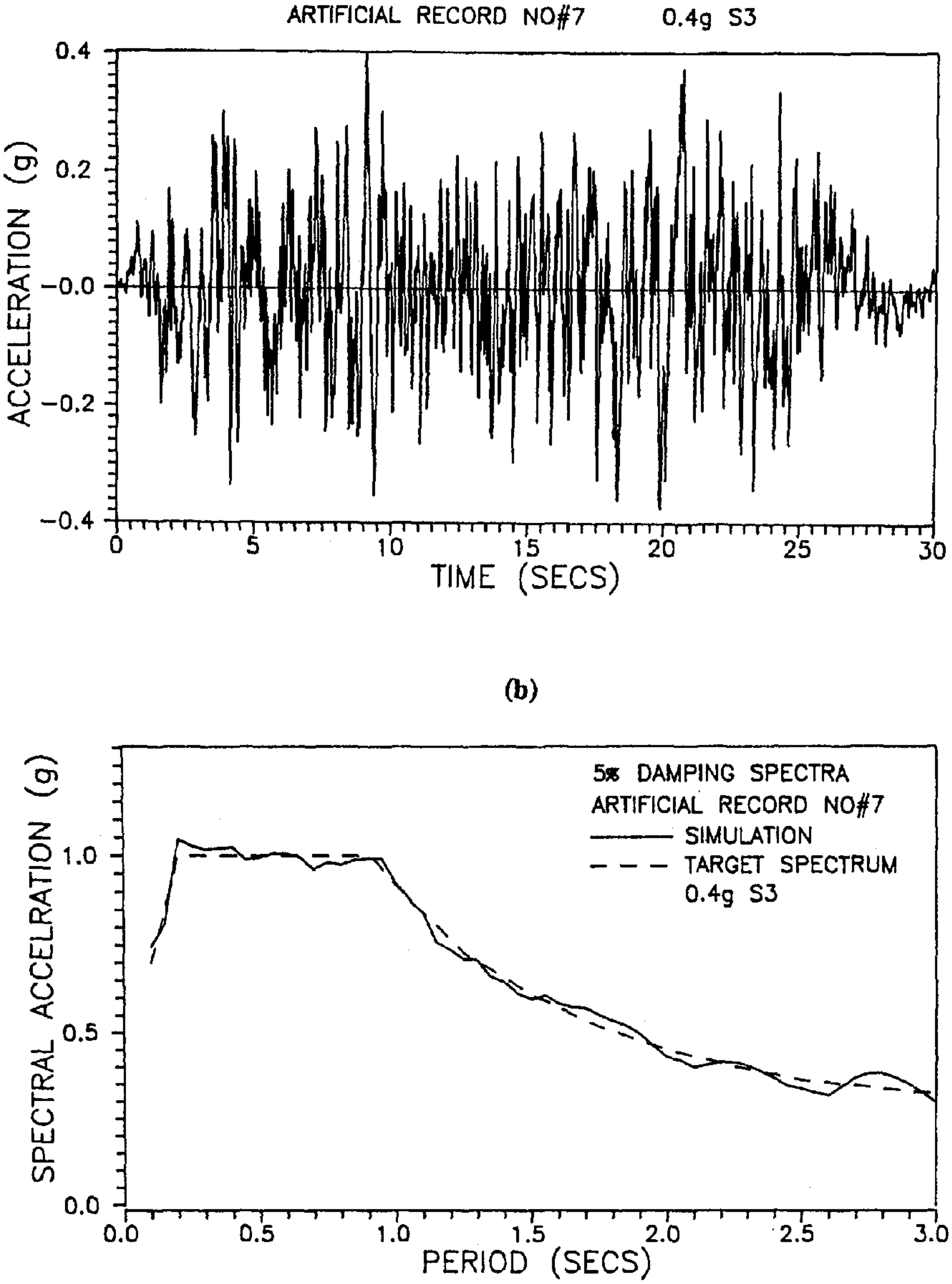
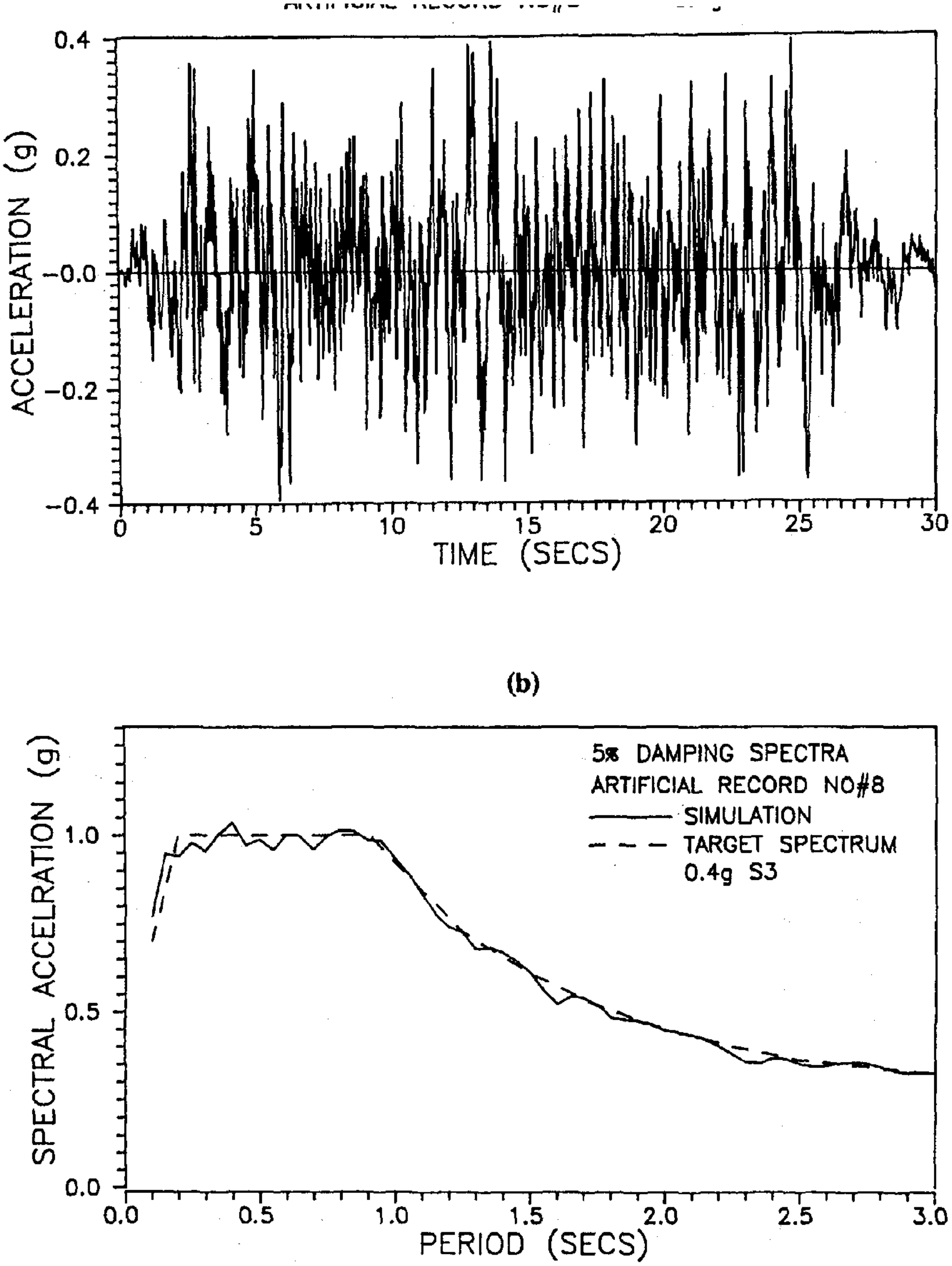


FIGURE 6-6 Artificial record No#6 compatible with 0.4g S2 Design Spectrum and comparison of its spectrum with the target spectrum.



(b)

FIGURE 6-7 Artificial record No#7 compatible with 0.4g S3 Design Spectrum and comparison of its spectrum with the target spectrum.



(b)

FIGURE 6-8 Artificial record No#8 compatible with 0.4g S3 Design Spectrum and comparison of its spectrum with the target spectrum.

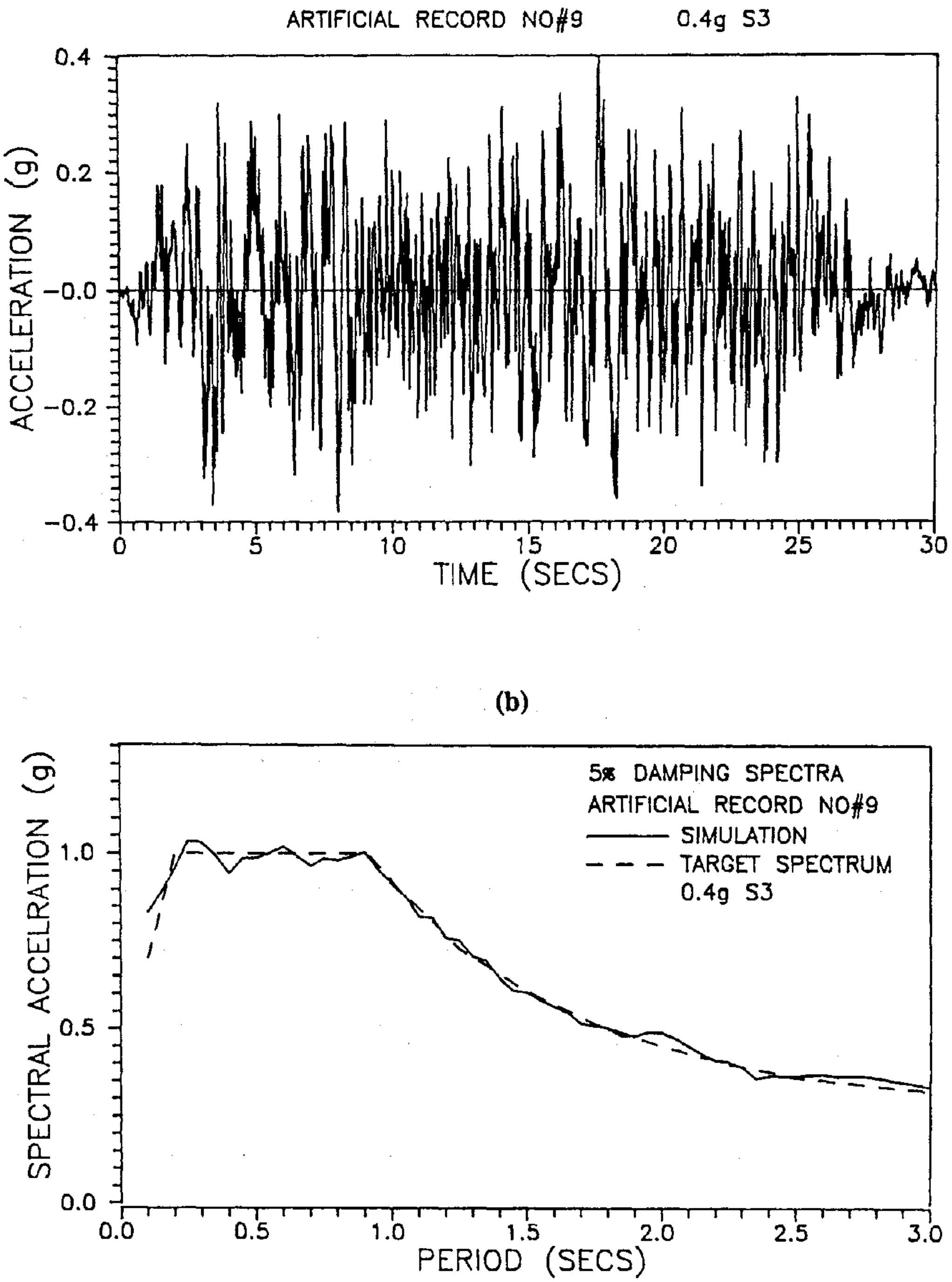


FIGURE 6-9 Artificial record No#9 compatible with 0.4g S3 Design Spectrum and comparison of its spectrum with the target spectrum.

TABLE 6.1 Summary of results of maximum base displacement at geometric center of 1 - story isolated structure excited in the transverse (T) direction by artificial records compatible to design spectra and comparison of these displacememts with the design displacements according to SEAOC static analysis procedure.

Soil Type	Sliding Isolation System Properties.								
	R=39.132 in (T_b= 2 sec.) fmax=0.10			R=88.048 in (T_b= 3 sec.) fmax=0.05			R=88.048 in (T_b= 3 sec.) fmax=0.10		
	Analysis (inch)	SEAOC (inch)	Ratio *	Analysis (inch)	SEAOC (inch)	Ratio *	Analysis (inch)	SEAOC (inch)	Ratio *
S1	1.43	2.81	1.97	3.44	5.28	1.53	1.96	3.11	1.59
S2	3.85	5.72	1.49	8.31	10.19	1.23	4.14	6.32	1.53
S3	6.92	9.06	1.31	9.73	16.31	1.68	6.53	10.55	1.62

* SEAOC/Analysis

TABLE 6.2 Summary of results of maximum base displacement at geometric center of 8 - story isolated structure excited in the transverse (T) direction by artificial records compatible to design spectra and comparison of these displacememts with the design displacements according to SEAOC static analysis procedure.

Soil Type	Sliding Isolation System Properties.								
	R=39.132 in (T_b = 2 sec.) fmax=0.10			R=88.048 in (T_b = 3 sec.) fmax=0.05			R=88.048 in (T_b = 3 sec.) fmax=0.10		
	Analysis (inch)	SEAOC (inch)	Ratio *	Analysis (inch)	SEAOC (inch)	Ratio *	Analysis (inch)	SEAOC (inch)	Ratio *
S1	2.27	2.81	1.24	3.48	5.28	1.52	2.64	3.11	1.18
S2	2.36	5.72	2.42	7.14	10.19	1.43	3.44	6.32	1.84
S3	7.97	9.06	1.14	13.63	16.31	1.20	8.69	10.55	1.21

* SEAOC/Analysis

TABLE 6.3 Analysis results for 1 - story isolated structure with R=39.132 in (T_b = 2 sec), f_{max} = 0.10, excited in transverse (T) direction by artificial record compatible with design spectra.

SOIL TYPE	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		*1st STORY SHEAR / WEIGHT*		*1st STORY DRIFT / HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
S1	ARTIFICIAL #1	0.00	1.42	0.02	1.43	1.01	0.000	0.133	0.000	0.140	0.000	0.0767
S1	ARTIFICIAL #2	0.00	1.42	0.02	1.44	1.01	0.000	0.132	0.000	0.130	0.000	0.0713
S1	ARTIFICIAL #3	0.00	1.43	0.02	1.43	1.01	0.000	0.133	0.000	0.150	0.000	0.0821
S2	ARTIFICIAL #4	0.00	3.19	0.01	3.20	1.00	0.000	0.179	0.000	0.142	0.000	0.0780
S2	ARTIFICIAL #5	0.00	3.85	0.02	3.87	1.01	0.000	0.195	0.000	0.170	0.000	0.0937
S2	ARTIFICIAL #6	0.00	2.23	0.02	2.25	1.01	0.000	0.154	0.000	0.180	0.000	0.0988
S3	ARTIFICIAL #7	0.00	4.97	0.02	4.98	1.00	0.000	0.225	0.000	0.157	0.000	0.0861
S3	ARTIFICIAL #8	0.00	6.92	0.02	6.93	1.00	0.000	0.273	0.000	0.163	0.000	0.0890
S3	ARTIFICIAL #9	0.00	5.67	0.02	5.68	1.00	0.000	0.243	0.000	0.193	0.000	0.1060

TABLE 6.4 Analysis results for 1 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.05, excited in transverse (T) direction by artificial record compatible with design spectra.

SOIL TYPE	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		1*st STORY SHEAR* / *WEIGHT*		1*st STORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
S1	ARTIFICIAL #1	0.00	3.26	0.01	3.27	1.00	0.000	0.085	0.000	0.084	0.000	0.0463
S1	ARTIFICIAL #2	0.00	2.78	0.01	2.78	1.00	0.000	0.080	0.000	0.107	0.000	0.0588
S1	ARTIFICIAL #3	0.00	3.44	0.01	3.45	1.00	0.000	0.088	0.000	0.081	0.000	0.0443
S2	ARTIFICIAL #4	0.00	8.31	0.01	8.32	1.00	0.000	0.144	0.000	0.115	0.000	0.0630
S2	ARTIFICIAL #5	0.00	6.52	0.01	6.52	1.00	0.000	0.123	0.000	0.098	0.000	0.0536
S2	ARTIFICIAL #6	0.00	6.34	0.01	6.34	1.00	0.000	0.121	0.000	0.098	0.000	0.0536
S3	ARTIFICIAL #7	0.00	8.15	0.01	8.15	1.00	0.000	0.141	0.000	0.127	0.000	0.0697
S3	ARTIFICIAL #8	0.00	7.37	0.01	7.37	1.00	0.000	0.132	0.000	0.080	0.000	0.0441
S3	ARTIFICIAL #9	0.00	9.73	0.01	9.73	1.00	0.000	0.160	0.000	0.096	0.000	0.0528

TABLE 6.5 Analysis results for 1 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0. 10, excited in transverse (T) direction by artificial record compatible with design spectra.

SOIL TYPE	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		1*st STORY SHEAR* / *WEIGHT*		1*st STORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
S1	ARTIFICIAL #1	0.00	1.63	0.02	1.65	1.01	0.000	0.115	0.000	0.134	0.000	0.0738
S1	ARTIFICIAL #2	0.00	1.56	0.02	1.59	1.02	0.000	0.116	0.000	0.132	0.000	0.0723
S1	ARTIFICIAL #3	0.00	1.96	0.02	1.99	1.02	0.000	0.119	0.000	0.150	0.000	0.0822
S2	ARTIFICIAL #4	0.00	2.93	0.02	2.96	1.01	0.000	0.131	0.000	0.147	0.000	0.0806
S2	ARTIFICIAL #5	0.00	4.14	0.03	4.17	1.01	0.000	0.145	0.000	0.134	0.000	0.0738
S2	ARTIFICIAL #6	0.00	2.50	0.02	2.52	1.01	0.000	0.125	0.000	0.180	0.000	0.0991
S3	ARTIFICIAL #7	0.00	5.01	0.02	5.01	1.00	0.000	0.155	0.000	0.155	0.000	0.0853
S3	ARTIFICIAL #8	0.00	6.53	0.02	6.54	1.00	0.000	0.172	0.000	0.147	0.000	0.0799
S3	ARTIFICIAL #9	0.00	6.29	0.04	6.30	1.00	0.000	0.170	0.000	0.145	0.000	0.0792

TABLE 6.6 Analysis results for 8 - story isolated structure with R=39.132 in (T_b = 2 sec), f_{max} = 0.10, excited in transverse (T) direction by artificial record compatible with design spectra.

SOIL TYPE	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		1*st STORY SHEAR* / *WEIGHT*		1*st STORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
S1	ARTIFICIAL #1	0.00	1.84	0.11	2.00	1.09	0.000	0.142	0.000	0.124	0.000	0.3020
S1	ARTIFICIAL #2	0.00	1.63	0.18	1.88	1.15	0.000	0.139	0.000	0.138	0.000	0.3275
S1	ARTIFICIAL #3	0.00	2.27	0.21	2.57	1.13	0.000	0.155	0.000	0.129	0.000	0.3059
S2	ARTIFICIAL #4	0.00	2.36	0.12	2.52	1.07	0.000	0.157	0.000	0.139	0.000	0.3321
S2	ARTIFICIAL #5	0.00	2.35	0.29	2.86	1.22	0.000	0.155	0.000	0.134	0.000	0.3160
S2	ARTIFICIAL #6	0.00	2.24	0.10	2.40	1.07	0.000	0.153	0.000	0.139	0.000	0.3308
S3	ARTIFICIAL #7	0.00	7.97	0.22	7.99	1.00	0.000	0.301	0.000	0.253	0.000	0.5973
S3	ARTIFICIAL #8	0.00	5.43	0.28	5.69	1.05	0.000	0.234	0.000	0.209	0.000	0.4952
S3	ARTIFICIAL #9	0.00	6.81	0.23	6.96	1.02	0.000	0.270	0.000	0.232	0.000	0.5433

TABLE 6.7 Analysis results for 8 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.05, excited in transverse (T) direction by artificial record compatible with design spectra.

SOIL TYPE	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	$\frac{\textit{BASE SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{1st STORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{1st STORY DRIFT}}{\textit{HEIGHT}}$ (%)	
		L	T	L	T		L	T	L	T	L	T
S1	ARTIFICIAL #1	0.00	3.48	0.13	3.68	1.06	0.000	0.088	0.000	0.092	0.000	0.2216
S1	ARTIFICIAL #2	0.00	2.40	0.12	2.47	1.03	0.000	0.075	0.000	0.080	0.000	0.1857
S1	ARTIFICIAL #3	0.00	3.31	0.18	3.55	1.07	0.000	0.086	0.000	0.082	0.000	0.1975
S2	ARTIFICIAL #4	0.00	7.14	0.14	7.21	1.01	0.000	0.129	0.000	0.127	0.000	0.2955
S2	ARTIFICIAL #5	0.00	5.68	0.16	5.97	1.05	0.000	0.113	0.000	0.108	0.000	0.2591
S2	ARTIFICIAL #6	0.00	6.41	0.17	6.52	1.02	0.000	0.120	0.000	0.119	0.000	0.2834
S3	ARTIFICIAL #7	0.00	9.15	0.15	9.32	1.02	0.000	0.152	0.000	0.138	0.000	0.3264
S3	ARTIFICIAL #8	0.00	8.33	0.19	8.58	1.03	0.000	0.143	0.000	0.147	0.000	0.3411
S3	ARTIFICIAL #9	0.00	13.63	0.16	13.69	1.00	0.000	0.203	0.000	0.192	0.000	0.4492

TABLE 6.8 Analysis results for 8 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.10, excited in transverse (T) direction by artificial record compatible with design spectra.

SOIL TYPE	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		*1st STORY SHEAR* / *WEIGHT*		*1st STORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
S1	ARTIFICIAL #1	0.00	2.24	0.14	2.46	1.10	0.000	0.121	0.000	0.122	0.000	0.2917
S1	ARTIFICIAL #2	0.00	1.64	0.20	1.92	1.17	0.000	0.117	0.000	0.133	0.000	0.3162
S1	ARTIFICIAL #3	0.00	2.64	0.25	3.02	1.14	0.000	0.129	0.000	0.126	0.000	0.3032
S2	ARTIFICIAL #4	0.00	2.87	0.20	3.12	1.09	0.000	0.131	0.000	0.121	0.000	0.2913
S2	ARTIFICIAL #5	0.00	2.38	0.24	2.76	1.16	0.000	0.121	0.000	0.135	0.000	0.3261
S2	ARTIFICIAL #6	0.00	3.44	0.17	3.62	1.05	0.000	0.136	0.000	0.127	0.000	0.3083
S3	ARTIFICIAL #7	0.00	7.84	0.32	8.02	1.02	0.000	0.188	0.000	0.168	0.000	0.3904
S3	ARTIFICIAL #8	0.00	6.81	0.30	7.10	1.04	0.000	0.176	0.000	0.165	0.000	0.3945
S3	ARTIFICIAL #9	0.00	8.69	0.39	8.81	1.01	0.000	0.196	0.000	0.180	0.000	0.4230

soil site, three identical analyses were performed corresponding to the three artificial records whose spectra match the design spectrum. The maximum value of the displacement of the isolation system calculated by those three analyses was the one of interest and is shown in Tables 6.1 and 6.2. Also listed in these tables are the corresponding design displacements according to SEAOC static analysis procedure, and the ratio between them and those estimated from the time history analyses of this study.

The SEAOC design formulae consistently overestimate the time history analysis results by an average factor of about 1.5. There is no special trend for a magnification or a reduction of the ratios of the displacements evaluated from the two approaches as a function of the soil type or isolation system properties. Rather, a random distribution of the ratio values is observed with respect to the soil type and the isolation system properties. It is interesting to note (see Tables 6.3 to 6.8) that the isolation system displacements in the 8 - story structure are either larger or smaller than the corresponding displacements of the 1 - story structure. Clearly, the flexibility of the superstructure has important effects on the response of the isolation system, particularly in the case of sliding isolation systems in which higher mode response occurs (Constantinou et al, 1990a and Mokha et al, 1990b).

Another important observation is that, essentially, the rotation of the base of the 1-story structure due to mass eccentricity is negligible. Tables 6.3 through 6.5 provide supplementary information for the 1-story structure where it can be seen that the ratio of the displacement between the corner bearing and the displacement at the center of mass of the base did not exceed the value of 1.02. At this point, it is interesting to refer to the results of the work done by Kircher and Lashkari, 1989, where for bilinear hysteretic behavior in the isolation system and for a 5% mass eccentricity and a rigid superstructure, corner bearing displacements up to 1.66 times the displacement at the center of mass were calculated. For the Friction Pendulum System, this behavior (which will also be discussed in other comparison approaches in Sections 7 and 8) indicates that, essentially, the resultant lateral force of the FPS bearings develops at the center of mass of the structure, thus no rotation occurs during an earthquake excitation.

This significant property is attributed to the fact that for each individual bearing, the developed lateral force is proportional to the weight carried by the bearing. (See equations 2.9, 4.6). Zayas et al 1987, has confirmed this behavior in shake table tests. The ratio of the corner bearing displacement to the displacement at the center of mass of the base is, according to SEAOC, given by equation 2.6. For the analyzed structure, this ratio is 1.24. When a more rational analysis is used, SEAOC allows the use of a smaller ratio which is not less than 1.1.

The property of the Friction Pendulum System to resist torsion becomes less apparent in the case of the 8-story structure (see Tables 6.6 through 6.8). A maximum ratio between the corner bearing displacement and the displacement at the center of mass of the base of 1.22 is observed. In the 1 - story structure, this ratio was only 1.02. An explanation for this difference is provided by the statement that in the 8 - story structure, an eccentricity between the mass center and the rigidity center existed in eight out of nine levels of the structure (at the isolation level there was no eccentricity since the lateral force developed at the FPS bearings was proportional to the axial load on the bearing). In the 1 - story structure, this eccentricity existed for the one out of two levels of the structure. In general, the eccentric inertia forces in the flexible superstructure result in torsional motion of the superstructure which "drives" the isolation system in similar motion. The SEAOC static design procedure does not account for the property of sliding isolation systems, and in particular the Friction Pendulum System, to reduce torsional effects due to mass eccentricity, especially for rigid or very stiff superstructures.

Finally, it is evident by observing Tables 6.3 through 6.8 that the excitation of both the structures in the transverse (T) direction resulted in the development of forces only in that direction. The occurrence of small values of displacements of the corner bearings in the longitudinal (L) direction (up to 0.04 inches for the 1-story structure and 0.39 inches for the 8-story structure) is due to the rotation of the base of the structure, whereas the displacement in the L direction at the center of mass of the two models and the respective developed shear forces were found in all cases to be zero.

Appendix A provides supplementary information for the results of the maximum calculated story shear force and interstory drift of all the floors, in the case of the 8 - story structure excited by the artificial earthquake motions. The interstory drift is divided by the story height (12ft). These results are particularly useful in studying the distribution of story shear with height of the structure, which is not attempted in this study. However, one could not avoid observing that the maximum story shear remains essentially the same in all stories except for the top two stories. This indicates higher mode response, a characteristic of sliding isolation systems which has been confirmed in experiments (Constantinou et al, 1990a and Mokha et al, 1990b). Furthermore, the interstory drift ratio is restricted to values less than 0.006, which satisfies the limits imposed by SEAOC.

6.2 Comparison of Time History Analysis to Bi-directional Excitation to SEAOC Design Formulae

To study the effect of bi-directional excitation on the response of isolated structures, the analyses reported in Section 6.1 are repeated with an additional excitation component in the longitudinal direction. The full simulated motion is applied in the transverse (T) direction and 83% of the same motion is applied in the longitudinal (L) direction. Detailed results are presented in Tables 6.9 to 6.11. Analyses were performed only for the 8-story structure.

As expected, the bi-directional excitation results in larger bearing displacements in both directions except in a single case, in which the opposite occurs. For comparison to the SEAOC design displacement, Table 6.12 was prepared. In this table, the maximum displacement among the three artificial records for each soil type is presented together with the SEAOC displacement and the ratio of this displacement to the calculated one. Evidently, for the considered bi-directional excitation, the time history results on the displacement are very close to the SEAOC values, which are on the conservative side within 25% of overestimation on the average.

The above results indicate that the SEAOC design formula can predict displacements within an acceptable range of overestimation provided that the earthquake excitation is interpreted as having bi-directional components. The two orthogonal components have the square root of the sum of the squares (SRSS) of their 5%-damped spectra matching the 1.3 times the 5% damped design spectrum ($\sqrt{1^2 + 0.83^2} = 1.3$). This is consistent with the dynamic time history analysis approach of SEAOC.

It is interesting to note that under the bi-directional artificial excitation, the ratio of corner bearing displacement to center point displacement is less than in the case of the one-directional excitation. In the considered bi-directional excitation, the two components are in phase, resulting in ground motion in a single direction at a 40 degree angle with respect to the longitudinal axis. With respect to this axis of excitation, the mass eccentricity is less than 8ft and equal to 6.13ft. This amounts to 3.8% rather than 5% mass eccentricity. This explains the reduction in torsion.

Concluding this section we note the following:

(1)The SEAOC displacement values are about 1.5 larger than those calculated in uni-directional artificial time history analyses.

(2)The SEAOC displacement values are about 1.25 larger than those calculated in bi-directional artificial time history analyses.

(3)The effect of bi-directional excitation appears to be significant. On the average, bi-directional excitation results in 20% larger response than uni-directional excitation. This difference is larger than the one observed in the study of Kircher and Lashkari, 1989. Responsible for this difference is the modeling of the isolation elements. In the Kircher and Lashkari, 1989 study, each isolator was modeled by two bilinear hysteretic elements placed at right angle. The interaction curve in this model is effectively square. In contrast, the model used in the present study has

TABLE 6.9 Analysis results for 8 - story isolated structure with R=39.132 in (T_b = 2 sec), f_{max} = 0.10, excited by bi-directional artificial record compatible with design spectra. 100% of artificial record in the transverse (T) and 83% of artificial record in the longitudinal (L) direction.

SOIL TYPE	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		*1st STORY SHEAR / WEIGHT*		*1st STORY DRIFT / HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
S1	ARTIFICIAL #1	1.79	2.16	1.87	2.30	1.06	0.105	0.128	0.104	0.125	0.2477	0.3009
S1	ARTIFICIAL #2	1.49	1.80	1.59	1.98	1.10	0.099	0.122	0.103	0.119	0.2403	0.2818
S1	ARTIFICIAL #3	2.12	2.54	2.26	2.80	1.10	0.116	0.141	0.098	0.119	0.2261	0.2781
S2	ARTIFICIAL #4	2.43	2.94	2.47	3.00	1.02	0.121	0.146	0.108	0.129	0.2518	0.3030
S2	ARTIFICIAL #5	2.18	2.56	2.44	3.03	1.18	0.116	0.140	0.099	0.120	0.2304	0.2814
S2	ARTIFICIAL #6	2.13	2.57	2.17	2.65	1.03	0.112	0.139	0.109	0.130	0.2557	0.3072
S3	ARTIFICIAL #7	6.08	7.31	6.10	7.35	1.00	0.216	0.264	0.179	0.219	0.4115	0.5067
S3	ARTIFICIAL #8	5.41	6.50	5.51	6.68	1.03	0.198	0.214	0.172	0.208	0.3942	0.4805
S3	ARTIFICIAL #9	6.23	7.50	6.30	7.62	1.02	0.218	0.266	0.194	0.232	0.4586	0.5510

TABLE 6.10 Analysis results for 8 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.05, excited by bi-directional artificial record compatible with design spectra. 100% of artificial record in the transverse (T) and 83% of artificial record in the longitudinal (L) direction.

SOIL TYPE	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		1*st STORY SHEAR* / *WEIGHT*		1*st STORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
S1	ARTIFICIAL #1	3.46	4.20	3.54	4.34	1.03	0.070	0.085	0.068	0.082	0.1634	0.1972
S1	ARTIFICIAL #2	2.65	3.18	2.69	3.26	1.03	0.060	0.072	0.059	0.071	0.1392	0.1691
S1	ARTIFICIAL #3	3.69	4.44	3.72	4.50	1.01	0.074	0.089	0.065	0.078	0.1517	0.1841
S2	ARTIFICIAL #4	7.10	8.54	7.13	8.58	1.01	0.112	0.134	0.108	0.130	0.2549	0.3082
S2	ARTIFICIAL #5	5.93	7.11	6.04	7.30	1.03	0.098	0.119	0.090	0.108	0.2104	0.2533
S2	ARTIFICIAL #6	6.52	7.84	6.57	7.93	1.01	0.105	0.126	0.101	0.121	0.2373	0.2877
S3	ARTIFICIAL #7	8.57	10.32	8.62	10.41	1.01	0.128	0.154	0.116	0.140	0.2701	0.3281
S3	ARTIFICIAL #8	8.73	10.49	8.81	10.62	1.01	0.130	0.157	0.120	0.144	0.2802	0.3393
S3	ARTIFICIAL #9	14.41	17.35	14.42	17.37	1.00	0.195	0.234	0.182	0.218	0.4246	0.5129

TABLE 6.11 Analysis results for 8 - story isolated structure with R=88.048 in ($T_b = 3$ sec), $f_{max} = 0.10$, excited by bi-directional artificial record compatible with design spectra. 100% of artificial record in the transverse (T) and 83% of artificial record in the longitudinal (L) direction.

SOIL TYPE	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		*1st STORY SHEAR* / *WEIGHT*		*1st STORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
S1	ARTIFICIAL #1	2.31	2.78	2.42	2.98	1.07	0.088	0.106	0.094	0.113	0.2246	0.2723
S1	ARTIFICIAL #2	1.55	1.86	1.68	2.09	1.12	0.079	0.098	0.092	0.106	0.2179	0.2535
S1	ARTIFICIAL #3	2.39	2.86	2.58	3.20	1.12	0.089	0.109	0.089	0.104	0.2124	0.2525
S2	ARTIFICIAL #4	3.44	4.16	3.48	4.22	1.01	0.100	0.121	0.102	0.122	0.2387	0.2861
S2	ARTIFICIAL #5	2.66	3.14	2.88	3.55	1.13	0.087	0.107	0.099	0.117	0.2376	0.2743
S2	ARTIFICIAL #6	2.90	3.50	3.01	3.71	1.06	0.094	0.116	0.096	0.114	0.2218	0.2666
S3	ARTIFICIAL #7	6.78	8.18	6.80	8.21	1.00	0.138	0.171	0.120	0.145	0.2842	0.3456
S3	ARTIFICIAL #8	6.00	7.31	6.15	7.48	1.02	0.130	0.159	0.122	0.145	0.2863	0.3445
S3	ARTIFICIAL #9	8.30	10.00	8.36	10.11	1.01	0.158	0.187	0.164	0.195	0.3830	0.4598

TABLE 6.12 Summary of results of maximum base displacement at geometric center of 8-story isolated structure excited by bi-directional artificial records compatible to design spectra (100% of artificial records in transverse (T) and 83% of artificial records in longitudinal (L) direction) and comparison of these displacements with the design displacements according to SEAOC static analysis procedure.

Soil Type	Sliding Isolation System Properties.								
	R=39.132 in (T_b = 2 sec.) fmax=0.10			R=88.048 in (T_b = 3 sec.) fmax=0.05			R=88.048 in (T_b = 3 sec.) fmax=0.10		
	Analysis (inch)	SEAOC (inch)	Ratio *	Analysis (inch)	SEAOC (inch)	Ratio *	Analysis (inch)	SEAOC (inch)	Ratio *
S1	2.54	2.81	1.11	4.44	5.28	1.19	2.86	3.11	1.09
S2	2.94	5.72	1.95	8.54	10.19	1.19	4.16	6.32	1.52
S3	7.50	9.06	1.21	17.35	16.31	0.94	10.00	10.55	1.06

* SEAOC/Analysis

circular interaction curve which closely resembles reality (Constantinou et al, 1990b). For bi-directional excitation the circular interaction curve results in larger displacement response than the square interaction curve.

SECTION 7

STATISTICAL EVALUATION OF RESPONSE FOR A SET OF RECORDED PAIRS OF HORIZONTAL EARTHQUAKE COMPONENTS.

In this section, a statistical approach for the evaluation of the nonlinear response of the structure-isolation models described in Section 4 is developed. This approach uses sets of time histories that are consistent in amplitude and frequency content with the design spectra currently required by the seismic codes, and calculates statistical quantities like mean values and standard deviations of the response parameters which are under consideration. In this way, a measure of the level of the inherent variation of the response parameters as a function of the variation in the ground motion is provided.

The idea of a statistical consideration and estimation of the results of a performed series of nonlinear analyses, for the prediction of response of sliding seismic isolated systems, is, as stated in Section 3, based on the work done by Kircher and Lashkari, 1989. In this work, a collection of nonlinear response data was created from where a basis for judging the validity and applicability of SEAOC design requirements was provided through a statistical processing. In the study examined in this section, all the major principles and assumptions that were included in the work of Kircher and Lashkari, 1989 are adopted.

7.1 Earthquake Time Histories.

The ground motions that were selected for this work are the same as those used by Kircher and Lashkari, 1989. According to them, appropriate earthquake time histories should be the ones that are consistent in amplitude and frequency content with the design spectra, currently required by seismic codes, i.e. the ATC-3 spectra. For this reason, the time histories were selected from the records that were used by Seed et al, 1974 to develop those site-depended spectra. It is interesting to note that Seed's study yielded results that have been used as the primary basis for the ATC-3 design spectra, and also for the seismic criteria of the Blue Book (SEAOC 1990b) and the Uniform Building Code (UBC, 1991).

The Seed study developed site-dependent spectra by calculating mean and mean-plus-one-standard-deviation spectra of normalized acceleration time history records. The earthquake records were grouped by one of four different site conditions, listed below with the corresponding ATC-3 soil type:

(1) Rock sites - Soil type S1.

(2)Stiff soils with depths less than about 150 ft. - Soil type S1.

(3)Deep cohesionless soils with depths greater than about 250 ft. - soil type S2.

(4)Soil deposits consisting of soft to medium stiff clays with associated strata of sands or gravels. - Soil type S3.

Horizontal earthquake records with peak ground acceleration (PGA) values of 0.05g or greater were selected by the Seed study from available data up to and including the San Fernando earthquake of 1971. The Seed study treated the two horizontal components as independent records and collected about 30 records each for rock, stiff soil and medium soil sites, and 15 records for soft soil sites.

In this study, only records with both horizontal components exceeding 0.10g PGA were considered appropriate for the nonlinear analyses. After elimination of the less significant records, the following number of records remained in each group:

(1) Rock sites - 10 pairs (20 records), representative of soil type S1.

(2) Stiff soil sites - 10 pairs (20 records), representative of soil type S1.

(3) Medium soil sites - 9 pairs (18 records), representative of soil type S2.

Since there were no records greater than 0.10g PGA for soft soil sites, this site condition was not evaluated during the series of the statistical evaluation analyses. Pertinent information for each pair of horizontal earthquake time histories is provided in Table 7.1 for records at rock sites, in Table 7.2 for records at stiff soil sites and in Table 7.3 for records at medium soil sites.

Values of the PGA and the peak ground velocity (PGV) given in these tables were taken directly from the California Institute of Technology data (CIT, 1974). In certain cases, it was noted that the PGA values reported in the Seed study differed from the CIT data. No explanation for these discrepancies could be found, except that some values reported in the Seed study may have been for "uncorrected" records. Each set of earthquake records has a large proportion from the San Fernando earthquake of 1971. The Seed study investigated the potential biasing of results that can occur if the spectra are dominated by the San Fernando earthquake, and concluded that the results were not unduly influenced. On the basis of the findings of Seed's study, Kircher and Lashkari, 1989 treated those earthquake records as representative of the ATC-3 design spectra for soil types S1 (rock and stiff soil types) and for soil types S2 (medium soil sites), respectively.

7.2 Scaling Factors

As referred in Section 2, for this study, Seismic Zone 4 was the only zone considered. The effective PGA for Seismic Zone 4 is 0.4 g, the value of acceleration specified by the ATC-3 study for scaling of the normalized response spectra. As summarized in Tables 7.1 through 7.3, the unscaled records have a variety of PGA values, most of which are considerably less than 0.4 g. Thus, scaling the records was required to insure that the response spectra be consistent with Seismic Zone 4 design spectra.

TABLE 7.1 Horizontal earthquake components recorded at rock sites.

No#	COMPONENT	EARTHQUAKE STATION No#	DATE	MAGNITUDE	DISTANCE FROM SOURCE (Km)	LOCAL MMI	DIRECTION	PGA (g)	PGV (in./sec)	PGD (in.)
1	T L	HELENA (323)	10/31/35	6.0	8	-	S00W S90W	0.146 0.145	2.89 -5.24	0.56 -1.47
2	T L	KERN COUNTY (095)	07/21/52	7.6	56	VII	S69E N21E	0.179 0.156	-6.97 -6.18	-3.60 -2.64
3	T L	LYTLE CREEK (290)	09/12/70	5.4	15	VI	S25W S65E	0.198 0.142	-3.79 -3.49	0.41 -0.87
4	T L	PARKFIELD (097)	06/27/66	5.6	7	VII	S25W N65W	-0.347 -0.269	8.87 -5.71	-2.16 1.83
5	T L	SAN FERNANDO (284)	02/09/71	6.6	30	VII	S08E S82W	0.217 0.202	3.89 2.46	2.76 -1.81
6	T L	SAN FERNANDO (126)	02/09/71	6.6	26	VII	S69E S21W	0.171 -0.146	-2.26 -3.38	0.49 0.69
7	T L	SAN FERNANDO (279)	02/09/71	6.6	3	IX	S16E S74W	-1.170 1.075	-44.57 -22.72	14.83 -4.26
8	T L	SAN FERNANDO (104)	02/09/71	6.6	40	VII	N87W N03E	-0.169 -0.140	-2.62 2.08	-2.33 -1.24
9	T L	SAN FERNANDO (128)	02/09/71	6.6	21	VIII	N21E N69W	-0.353 0.283	5.81 -5.02	0.70 3.49
10	T L	SAN FERNANDO (220)	02/09/71	6.6	24	VII	N00E S90W	-0.167 0.150	4.88 5.91	-1.91 -2.14

TABLE 7.2 Horizontal earthquake components recorded at stiff soil sites.

No#	COMPONENT	EARTHQUAKE STATION No#	DATE	MAGNITUDE	DISTANCE FROM SOURCE (Km)	LOCAL MMI	DIRECTION	PGA (g)	PGV (in./sec)	PGD (in.)
11	T L	LOWER CA (117)	12/30/34	6.5	58	-	S90W S00W	-0.183 -0.160	4.55 -8.21	-1.44 -1.65
12	T L	IMPERIAL VALEY EL CENTRO (117)	05/18/40	6.6	8	VIII	S00E S90W	0.348 0.214	13.17 -14.53	4.28 -7.79
13	T L	PARKFIELD (014)	06/27/66	5.6	5	VIII	N85E N05W	-0.434 -0.355	-10.02 -9.12	-2.80 -2.09
14	T L	SAN FERNANDO (110)	02/09/71	6.6	21	VIII	N21E N69W	-0.315 -0.270	-6.76 -10.94	1.67 3.73
15	T L	SAN FERNANDO (135)	02/09/71	6.6	35	VII	N90E S00W	-0.211 0.170	-8.32 -6.50	-5.79 3.17
16	T L	SAN FERNANDO (208)	02/09/71	6.6	39	VII	N00E S90W	-0.136 0.144	8.79 7.30	4.50 -4.57
17	T L	SAN FERNANDO (211)	02/09/71	6.6	39	VII	NORTH WEST	0.157 -0.132	-6.90 8.45	3.17 -4.57
18	T L	SAN FERNANDO (466)	02/09/71	6.6	28	VII	N11E N79W	0.225 -0.149	-11.12 -9.23	-5.30 -4.06
19	T L	SAN FERNANDO (253)	02/09/71	6.6	28	VII	S12W N78W	-0.248 0.201	12.43 -7.01	7.21 3.73
20	T L	SAN FERNANDO (199)	02/09/71	6.6	39	VII	N90E SOOW	-0.165 -0.161	-6.54 7.24	-4.08 3.56

TABLE 7.3 Horizontal erthquake components recorded at medium soil sites.

No#	COMPONENT	EARTHQUAKE STATION No#`	DATE	MAGNITUDE	DISTANCE FROM SOURCE (Km)	LOCAL MMI	DIRECTION	PGA (g)	PGV (in./sec)	PGD (in.)
21	T L	WESTERN WASH (325)	04/13/49	7.1	20	VIII	N86E N04W	-0.280 0.165	-6.72 8.43	4.08 -3.38
22	T L	EUREKA (022)	12/21/54	6.5	25	VIII	N79E N11W	-0.258 0.168	11.56 -12.44	5.53 4.88
23	T L	EUREKA (023)	12/21/54	6.5	30	VII	N46W N44E	0.201 0.159	-10.24 14.03	-3.79 -5.58
24	T L	FERNDALE (023)	12/10/67	5.6	25	VII	S44W N46W	-0.237 0.105	4.70 4.65	0.65 -0.67
25	T L	SAN FERNANDO (241)	02/09/71	6.6	16	VII	N00W S90W	-0.255 -0.134	-11.81 9.42	-5.87 5.45
26	T L	SAN FERNANDO (458)	02/09/71	6.6	19	VII	SOOW S90W	0.116 0.105	12.46 -11.33	-6.93 6.02
27	T L	SAN FERNANDO (264)	02/09/71	6.6	30	VII	N00E N90E	-0.201 -0.185	-3.87 -6.47	-1.07 2.72
28	T L	SAN FERNANDO (267)	02/09/71	6.6	30	VII	S82E S08W	0.212 0.142	5.48 3.62	-1.95 -1.14
29	T L	PUGET SOUND (325)	04/29/65	6.5	58	VII	S86W S04E	-0.198 0.137	5.14 3.21	-1.51 -1.07

Two methods were used to scale time histories: scaling by peak ground acceleration (PGA) and scaling by peak ground velocity (PGV). The first method, which parallels the approach taken by the Seed study, scaled each pair of earthquake components by a common factor such that the average PGA of the two components is equal to 0.4g. This method is consistent with the approach used by the Seed study except that the Seed study scaled (i.e. normalized) each component individually, rather than in pairs.

The advantage of using PGA to scale the records is that it is the same method used implicitly in the Seed study to develop site-dependent design spectra. The shortcoming of scaling the time histories by PGA is that the response of an isolated structure is primarily influenced by the amplitude and frequency content of the velocity domain of the design spectrum. As a second method for scaling, each pair of earthquake components was scaled by a common factor such that the average PGV of the two components was equal to either 12 in./sec for rock site, 18 in./sec for stiff soil sites or 22.5 in./sec for medium soil sites. The same values were adopted in the work done by Kircher and Lashkari, 1989. Scaling the records by PGV, rather than by PGA was considered a more appropriate method of representing the amplitude and frequency content of ground motion at periods greater than 1.0 second. The scaling factors used in the series of the statistical evaluation analyses are listed in Table 7.4.

Figures 7-1 to 7-3, show, grouped by the three soil types, the average spectra of PGV-scaled and PGA-scaled time histories in the longitudinal (L), transverse (T) directions and the spectra that are created by averaging the square root of the sum of the squares of the spectra of the two horizontal components of every earthquake, individually ($(L^2+T^2)^{1/2}$). Also, each figure shows (with solid line) the respective design spectrum, which is one of the three ATC-3 Design Spectra (see Figure 2-1). An observation can be made to the fact that the average spectra of the time histories match the design spectra reasonably well for stiff soil and medium soil sites. For rock sites, however, the average acceleration spectra are lower than the ATC-3 design criteria for long periods. The reason for this large discrepancy between the design criteria of the ATC-3 study and the average spectrum for rock sites is not known.

TABLE 7.4 Scaling factors of the earthquake components.

No#	EARTHQUAKE STATION No#	PGA SCALED	PGV SCALED
1	HELENA (323)	2.749	2.952
2	KERN COUNTY (095)	2.388	1.825
3	LYTLE CREEK (290)	2.353	3.297
4	PARKFIELD (097)	1.299	1.646
5	SAN FERNANDO (284)	1.909	3.780
6	SAN FERNANDO (126)	2.524	4.255
7	SAN FERNANDO (279)	0.356	0.357
8	SAN FERNANDO (104)	2.589	5.106
9	SAN FERNANDO (128)	1.258	2.216
10	SAN FERNANDO (220)	2.524	2.224
11	LOWER CA (117)	2.332	2.821
12	IMP. VALLEY EL CENTRO (117)	1.423	1.300
13	PARKFIELD (014)	1.014	1.881
14	SAN FERNANDO (110)	1.368	2.034
15	SAN FERNANDO (135)	2.100	2.429
16	SAN FERNANDO (208)	2.857	2.237
17	SAN FERNANDO (211)	2.768	2.345
18	SAN FERNANDO (466)	2.139	1.769
19	SAN FERNANDO (253)	1.782	1.852
20	SAN FERNANDO (199)	2.454	2.612
21	WESTERN WASH (325)	1.798	2.970
22	EUREKA (022)	1.878	1.875
23	EUREKA (023)	2.222	1.854
24	FERNDALE (023)	2.339	4.813
25	SAN FERNANDO (241)	2.057	2.120
26	SAN FERNANDO (458)	3.620	1.892
27	SAN FERNANDO (264)	2.073	4.352
28	SAN FERNANDO (267)	2.260	4.945
29	PUGET SOUND (325)	2.388	5.389

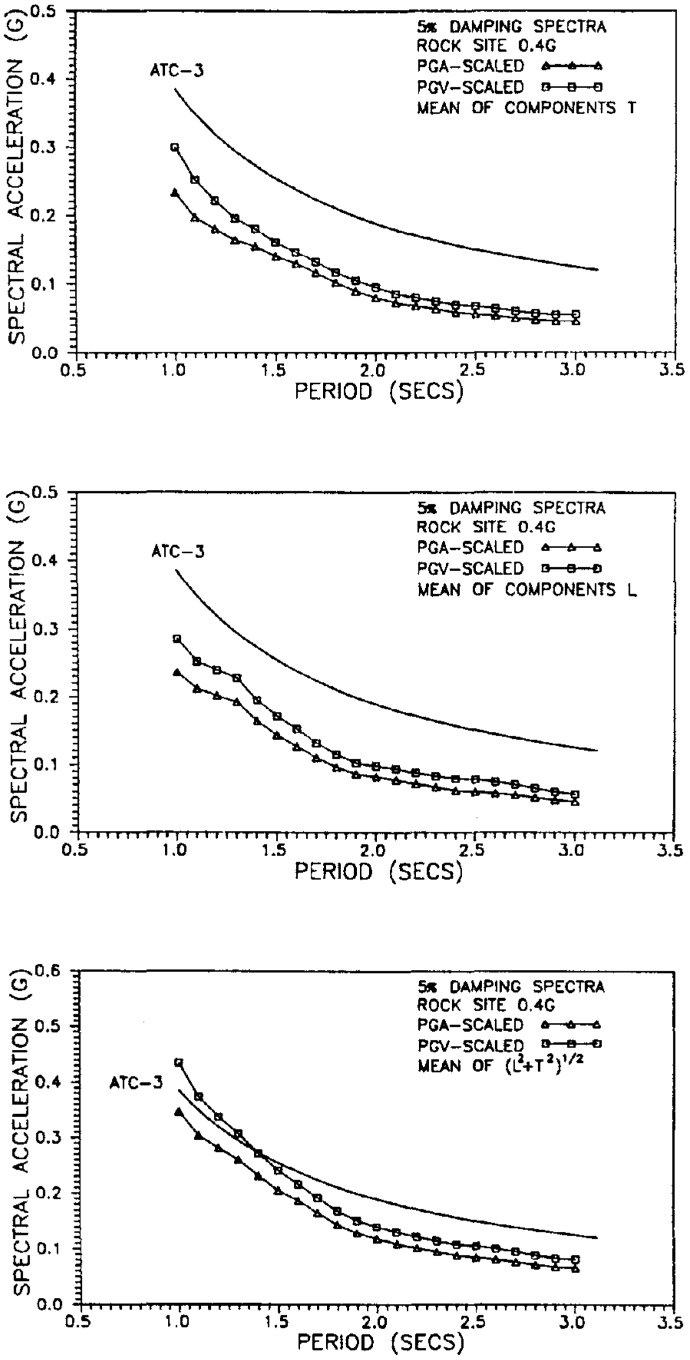


FIGURE 7-1 5% average spectra of components T,L and square root of sum of square of components T and L of PGV and PGA scaled motions used in dynamic analyses and comparison with the Design Spectrum. Earthquake records recorded at Rock Sites.

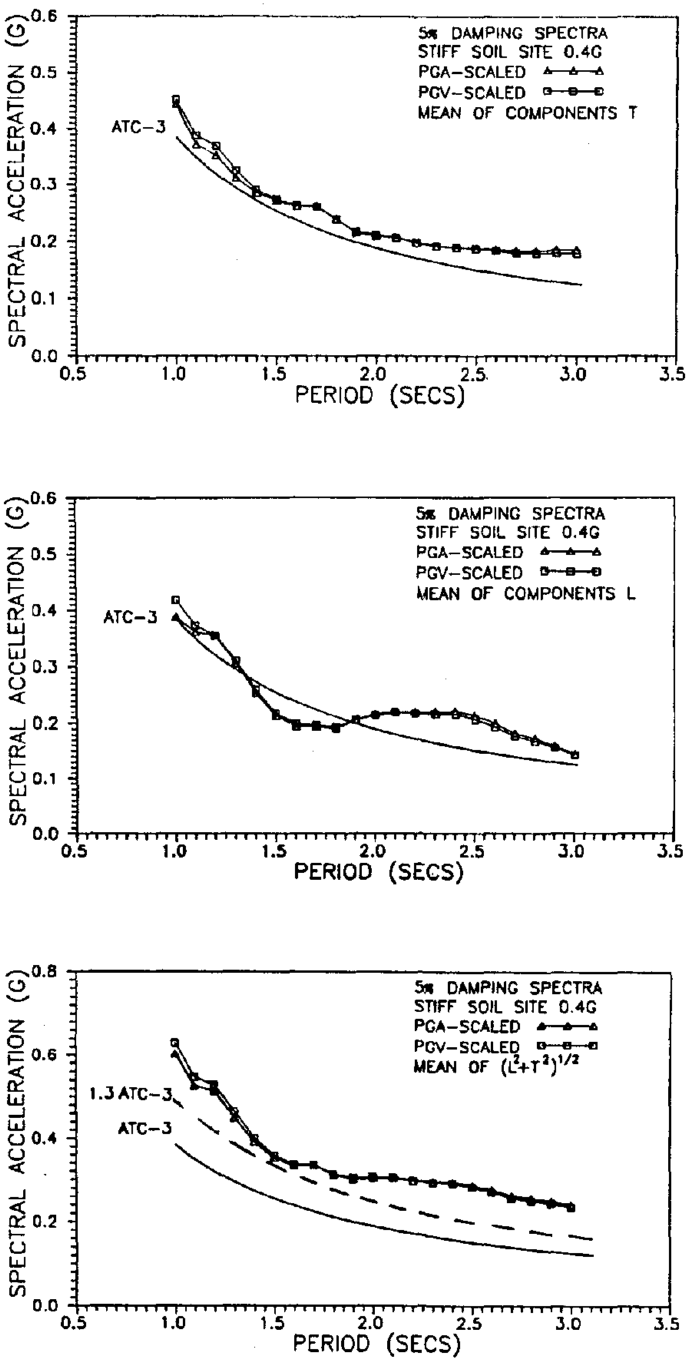


FIGURE 7-2 5% average spectra of components T,L and square root of sum of squares of components T and L of PGV and PGA scaled motions used in dynamic analyses and comparison with the Design Spectrum. Earthquake records recorded at Stiff Soil Sites.

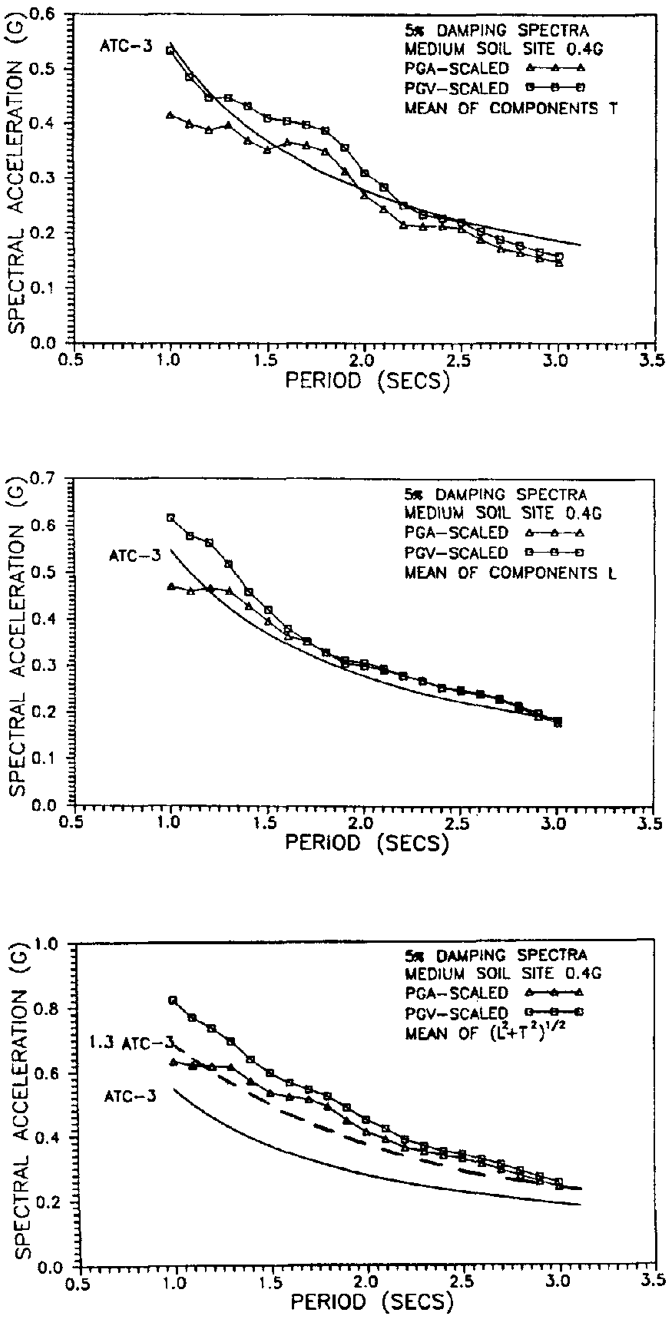


FIGURE 7-3 5% average spectra of components T,L and square root of sum of squares of components T and L of PGV and PGA scaled motions used in dynamic analyses and comparison with the Design Spectrum. Earthquake records recorded at Medium Soil Sites.

Apparently, the ATC-3 study felt that the time histories used by the Seed study to develop rock spectra do not contain a level of ground motion appropriate for design of long-period structures. Regardless of the motive, the ATC-3 study defined rock and stiff soil as a single site condition (soil type S1) and based the design criteria for this site condition of the average spectrum of stiff soil time histories.

Note that in Figures 7-1 to 7-3, the mean of the square root of the sum of the squares of the spectra values of the two horizontal components ($(L^2+T^2)^{1/2}$) of the cases of stiff soil and medium soil sites is substantially larger than the design spectrum. Even when the mean of $(L^2+T^2)^{1/2}$ spectra is compared to 1.3 times the design spectrum (doted line), still the $(L^2+T^2)^{1/2}$ spectra indicate a stronger motion particularly for stiff soil (Figure 7-2). The response spectra for each of the components of the PGV scaled motions are presented in Appendix B. It may be observed that the response spectra of some of these motions exhibit strong distinct peaks in the range of periods of 1.4 to 1.7 seconds (motions No. 15,18,19,23 and 25). One would expect that such peaks are characteristics of motions recorded on soil types other than S1 or S2. Actually, these motions resulted in bearing displacements which are considerably larger than those for other motions within each soil group. When these motions are removed from the sample, their mean $(L^2+T^2)^{1/2}$ spectra appear to be consistent with the 1.3 times the design spectrum for stiff and medium soil sites.

7.3 Comparison of Time History Analysis Results to SEAOC Design Formulae

In the previous section, two methods were used in this study to scale time histories: scaling by peak ground acceleration (PGA) and scaling by peak ground velocity (PGV). The analysis results have shown that the estimated mean values for the displacement of the isolation system according to the two methods were predicted to be almost the same whereas the standard deviation values differed thoroughly. When scaling by PGA, the standard deviations were as much as two times greater than the standard deviations estimated through the PGV scaling. It was concluded that the results based on PGA scaling were less representative of the behavior of isolated structures. Accordingly, they are not reported in this study.

Tables 7.5 to 7.22 present the results of analyses of the six structure/isolation systems (1 - story and 8 - story with three different isolation system properties) to the 29 pairs of PGV scaled motions. The tables include the peak displacement in the longitudinal (L) and transverse (T) directions at the base center and at the corner bearing, the peak base shear and first story shear (normalized by the total weight of the structure) and the peak first story drift ratio (for story height of 12 ft). Results on the story shear and the interstory drift ratio for the other stories of the 8 - story structure are presented in Appendix C. Furthermore, the tables present values of the ratio of peak corner displacement to peak base center displacement, as well as means and standard deviations (σ) of the calculated response quantities. For the comparison to the SEAOC design procedure for the displacement of the isolation system, the quantities of interest are the mean and standard deviation of the maximum displacement between components L and T for each of the three groups of the soil conditions. These values are listed in the last two lines of each table. For soil conditions of stiff and medium soils, certain motions have been excluded in the calculation of the mean and standard deviation (value in parenthesis and identified by an asterisk). These motions were those having in their spectra distinct peaks at high values of period. As discussed earlier, these motions may not be representative of stiff and medium soil conditions, but rather representative of soils with deeper profiles.

The first observation to be made in the results of Tables 7.5 to 7.22 is that the corner to center displacement ratio is equal to unity for the 1 - story structure, (mean = 1, σ = 0). This is significantly different than the value of 1.24 (by use of equation 2.6) required by the SEAOC static procedure or the minimum 1.1 value allowed when proper analysis is performed. In the case of the 8 - story structure, the corner displacement is larger than in the case of the

1 - story structure. Evidently, the torsional response is affected more by the flexibility of the superstructure and the properties of the isolation system than by the plan dimensions of the building.

An other observation to be made is the fact that the peak displacements of the isolated structure are significantly influenced by local site conditions. In general, peak response differs between rock, stiff soil and medium soil sites in a manner consistent with the differences in the mean spectra of the time histories of the different sites (see Figures 7-1 through 7-3).

For comparison of the calculated values of the isolation system displacement to the SEAOC static procedure, Tables 7.23 and 7.24 are presented. They include the mean and the mean plus one standard deviation values of the maximum displacement which occur in either the longitudinal (L) or transverse (T) direction. The tables also include the SEAOC minimum design values. Furthermore, Tables 7.25 and 7.26 present the same information but with certain records removed from the sample as not being representative of the assumed soil conditions. The reported values in Tables 7.25 and 7.26 are those in Tables 7.5 to 7.22 which are included in parenthesis and identified by an asterisk.

From the results of Tables 7.23 and 7.24, it may be observed that the SEAOC formula for the design displacement can predict well or accurately well the mean estimated values when the excitation is referring to stiff or medium soil sites. In general, the design values for those site conditions are between the mean and the mean plus one standard deviation of the estimated values or slightly lower than the mean values. However, for rock sites, the design displacements of SEAOC are consistently higher than the ones predicted through the dynamic analyses. Of course, this is primarily attributed to the fact that SEAOC specifies the same design displacement for rock and stiff soil sites since both of them are corresponding to soil type S1.

From the results of Tables 7.25 and 7.26 it may be observed that the mean values are slightly lower than the ones described in Tables 7.23 and 7.24. However, the standard deviation values are significantly lower than the originally estimated. Both sets of results are presented because together they provide a better picture of the variation in response due to the inherent variability of ground motion.

Based on the results of Tables 7.23 to 7.26 it may concluded that the SEAOC displacements are in good agreement with the mean of the peak displacements as calculated in the nonlinear dynamic analysis. A key point to be made is that the above conclusion is based on the results of analyses with bi-directional excitation and with circular interaction curve for the isolation bearing model. As explained in Section 6, the combination of these two factors results in larger bearing displacements than when a square interaction curve model is used.

To quantify the effect of the isolation bearing model, the analysis of the 1-story isolated structure with $R = 39.132$ in ($T_b = 2$ sec), $f_{max} = 0.10$ and excited by PGV scaled earthquake motions recorded on medium soil sites was repeated. Each isolation bearing was modeled by two bilinear hysteretic elements placed along the T and L directions of the structure. Each element had force-displacement characteristics described by equations 4.6 and 4.7. Effectively, the interaction curve between the forces in the two orthogonal directions was square. A comparison of this model to the previously used circular interaction model is presented in Figure 7-4. As seen in this figure the square interaction model results in force-displacement relation which is dependent on the direction of motion. The force is always larger than that in the circular interaction model.

The results of the analysis of the 1-story structure are summarized in Table 7.27. This table should be compared to Table 7.7 which contains results for the same structure but with circular interaction model for the bearings. The results clearly demonstrate that the square interaction model predicts bearing displacements which are about 17% less than those predicted by the circular interaction model.

When comparing to the SEAOC static procedure, SEAOC predicts 5.72 in. displacement (see Table 7.23) as compared to the mean of 4.95 in. of the analysis with square interaction model.

TABLE 7.5 **Analysis results for 1 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.**

EXCITATION		BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		*1st STORY SHEAR* / *WEIGHT*		*1st STORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
1	HELENA (323)	2.46	0.86	2.46	0.87	1.00	0152.	0.099	0.136	0.086	0.0742	0.0469
2	KERN COUNTY (095)	1.51	2.16	1.51	2.17	1.00	0.131	0.119	0.108	0.118	0.0585	0.0648
3	LYTLE CREEK (290)	1.95	1.28	1.95	1.29	1.00	0.140	0.116	0.108	0.105	0.0583	0.0579
4	PARKFIELD (097)	1.18	0.99	1.18	1.00	1.00	0.123	0.111	0.089	0.124	0.0481	0.0684
5	SAN FERNANDO (284)	0.95	0.88	0.95	0.88	1.00	0.102	0.113	0.187	0.222	0.1012	0.1217
6	SAN FERNANDO (126)	2.19	1.61	2.20	1.61	1.00	0.140	0.128	0.145	0.162	0.0784	0.0891
7	SAN FERNANDO (279)	1.36	1.93	1.36	1.93	1.00	0.116	0.129	0.107	0.144	0.0583	0.0791
8	SAN FERNANDO (104)	0.95	1.51	0.96	1.52	1.01	0.115	0.131	0.173	0.184	0.0938	0.1011
9	SAN FERNANDO (128)	1.38	1.15	1.38	1.15	1.00	0.133	0.126	0.159	0.179	0.0862	0.0979
10	SAN FERNANDO (220)	1.99	0.90	2.00	0.91	1.01	0.144	0.099	0.126	0.133	0.0683	0.0728

MEAN	1.59	1.33	1.59	1.33	1.00	0.130	0.117	0.134	0.146	0.0725	0.0799
σ	0.50	0.44	0.50	0.44	0.00	0.015	0.011	0.030	0.039	0.0165	0.0215
MEAN OF MAX (L,T)	1.77		1.77			0.134		0.151		0.0827	
σ OF MAX(L,T)	0.46		0.46			0.010		0.034		0.0185	

TABLE 7.6 Analysis results for 1 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		*1st STORY SHEAR / WEIGHT*		*1st STORY DRIFT / HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
11	LOWER CA (117)	3.00	1.72	3.00	1.72	1.00	0.152	0.125	0.132	0.129	0.0715	0.0706
12	EL CENTRO (117)	2.23	2.36	2.23	2.37	1.00	0.152	0.143	0.101	0.113	0.0548	0.0620
13	PARKFIELD (014)	1.07	1.31	1.08	1.31	1.00	0.117	0.121	0.110	0.113	0.0597	0.0618
14	SAN FERNANDO (110)	4.07	2.14	4.08	2.14	1.00	0.182	0.150	0.136	0.151	0.0737	0.0827
15	SAN FERNANDO (135)	1.59	4.86	1.59	4.87	1.00	0.107	0.213	0.116	0.144	0.0628	0.0787
16	SAN FERNANDO (208)	1.31	1.93	1.31	1.93	1.00	0.112	0.136	0.109	0.127	0.0590	0.0697
17	SAN FERNANDO (211)	1.70	1.72	1.70	1.72	1.00	0.116	0.132	0.096	0.108	0.0520	0.0596
18	SAN FERNANDO (466)	1.10	1.89	1.10	1.89	1.00	0.113	0.139	0.120	0.143	0.0649	0.0783
19	SAN FERNANDO (253)	1.05	1.99	1.05	2.00	1.00	0.117	0.141	0.121	0.145	0.0658	0.0792
20	SAN FERNANDO (199)	2.07	1.71	2.07	1.71	1.00	0.132	0.121	0.119	0.147	0.0645	0.0808
MEAN		1.92	2.16	1.92	2.17	1.00	0.130	0.142	0.116	0.132	0.0629	0.0723
σ		0.93	0.94	0.93	0.94	0.00	0.023	0.025	0.012	0.015	0.0064	0.0083
MEAN OF MAX (L,T)		2.52		2.52			0.150		0.132		0.0724	
σ OF MAX(L,T)		1.07		1.07			0.026		0.015		0.0083	

TABLE 7.7 Analysis results for 1 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Weight is total weight of structure including base. Height is 12ft.

EXCITATION		BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		*1st STORY SHEAR / WEIGHT*		*1st STORY DRIFT / HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
21	WESTERN WASH (325)	3.31	3.01	3.31	3.01	1.00	0.168	0.156	0.152	0.153	0.0791	0.0837
22	EUREKA (022)	2.98	2.70	2.98	2.70	1.00	0.151	0.158	0.091	0.115	0.0492	0.0630
23	EUREKA (023)	11.42	6.63	11.42	6.63	1.00	0.361	0.262	0.183	0.139	0.0996	0.0763
24	FERNDALE (023)	4.30	3.58	4.30	3.58	1.00	0.162	0.175	0.129	0.121	0.0702	0.0661
25	SAN FERNANDO (241)	4.82	5.83	4.82	5.83	1.00	0.215	0.227	0.111	0.145	0.0600	0.0796
26	SAN FERNANDO (458)	5.68	3.16	5.68	3.16	1.00	0.232	0.176	0.126	0.099	0.0685	0.0543
27	SAN FERNANDO (264)	6.80	3.84	6.80	3.84	1.00	0.237	0.134	0.145	0.125	0.0788	0.0688
28	SAN FERNANDO (267)	4.01	4.26	4.01	4.26	1.00	0.197	0.204	0.138	0.163	0.0748	0.0892
29	PUGET SOUND (325)	2.03	8.84	2.03	8.84	1.00	0.140	0.316	0.133	0.186	0.0721	0.1017

MEAN	5.04	4.65	5.04	4.65	1.00	0.207	0.201	0.134	0.138	0.0725	0.0759
σ	2.63	1.93	2.63	1.93	0.00	0.064	0.055	0.024	0.025	0.0131	0.0137
MEAN OF MAX (L,T)	5.94 (5.25) *		5.94 (5.25) *			0.231		0.149		0.0816	
σ OF MAX(L,T)	2.59 (1.82) *		2.59 (1.82) *			0.064		0.023		0.0127	

* Values in parentheses are estimated without the contribution of earthquake excitation No# 23.

TABLE 7.8 Analysis results for 1 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		*1st STORY SHEAR* / *WEIGHT*		*1st STORY DRFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
1	HELENA (323)	3.81	0.99	3.81	0.99	1.00	0.088	0.054	0.055	0.060	0.0301	0.0330
2	KERN COUNTY (095)	2.49	2.90	2.49	2.90	1.00	0.076	0.077	0.055	0.056	0.0299	0.0307
3	LYTLE CREEK (290)	1.82	1.63	1.82	1.64	1.01	0.065	0.059	0.063	0.055	0.0342	0.0299
4	PARKFIELD (097)	1.55	1.42	1.55	1.42	1.00	0.065	0.061	0.051	0.069	0.0276	0.0378
5	SAN FERNANDO (284)	1.17	2.44	1.17	2.44	1.00	0.061	0.071	0.104	0.111	0.0562	0.0605
6	SAN FERNANDO (126)	3.33	1.76	3.33	1.76	1.00	0.078	0.068	0.082	0.086	0.0444	0.0473
7	SAN FERNANDO (279)	2.15	3.89	2.15	3.89	1.00	0.072	0.081	0.056	0.077	0.0302	0.0420
8	SAN FERNANDO (104)	1.43	2.52	1.43	2.52	1.00	0.064	0.077	0.101	0.094	0.0550	0.0519
9	SAN FERNANDO (128)	1.62	1.53	1.63	1.53	1.01	0.066	0.066	0.097	0.102	0.0529	0.0556
10	SAN FERNANDO (220)	3.30	1.66	3.31	1.66	1.00	0.079	0.063	0.068	0.071	0.0371	0.0393
	MEAN	2.27	2.07	2.27	2.07	1.00	0.071	0.068	0.073	0.078	0.0398	0.0428
	σ	0.88	0.82	0.88	0.82	0.00	0.008	0.008	0.020	0.019	0.0108	0.0101
	MEAN OF MAX (L,T)	2.72		2.72			0.075		0.080		0.0435	
	σ OF MAX(L,T)	0.82		0.82			0.007		0.018		0.0100	

TABLE 7.9 Analysis results for 1 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		1*st STORY SHEAR* / *WEIGHT*		1*st STORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
11	LOWER CA (117)	4.07	2.06	4.07	2.06	1.00	0.091	0.066	0.066	0.063	0.0357	0.0347
12	EL CENTRO (117)	7.73	5.42	7.73	5.42	1.00	0.135	0.105	0.073	0.066	0.0394	0.0364
13	PARKFIELD (014)	1.48	2.41	1.48	2.42	1.00	0.064	0.075	0.048	0.073	0.0258	0.0398
14	SAN FERNANDO (110)	5.74	3.28	5.74	3.28	1.00	0.097	0.086	0.064	0.062	0.0347	0.0342
15	SAN FERNANDO (135)	4.26	11.66	4.26	11.66	1.00	0.091	0.178	0.071	0.095	0.0385	0.0520
16	SAN FERNANDO (208)	4.33	5.67	4.33	5.67	1.00	0.091	0.106	0.061	0.065	0.0331	0.0357
17	SAN FERNANDO (211)	5.03	5.00	5.03	5.00	1.00	0.096	0.091	0.059	0.061	0.0320	0.0337
18	SAN FERNANDO (466)	7.07	10.86	7.07	10.86	1.00	0.128	0.168	0.068	0.085	0.0371	0.0466
19	SAN FERNANDO (253)	5.34	11.57	5.34	11.57	1.00	0.082	0.172	0.071	0.087	0.0390	0.0478
20	SAN FERNANDO (199)	4.22	2.69	4.22	2.70	1.00	0.082	0.072	0.067	0.083	0.00365	0.0453

MEAN	4.93	6.06	4.93	6.06	1.00	0.096	0.112	0.065	0.074	0.0352	0.0406
σ	1.65	3.67	1.65	3.67	0.00	0.020	0.042	0.007	0.012	0.0039	0.0064
MEAN OF MAX (L,T)	6.90 (4.98) *		6.90 (4.98) *			0.120		0.075		0.0411	
σ OF MAX(L,T)	3.20 (1.54) *		3.20 (1.54) *			0.038		0.011		0.061	

* Values in parentheses are estimated without the contribution of earthquake excitations No# 15, 18, 19.

TABLE 7.10 Analysis results for 1 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Weight is total weight of structure including base. Height is 12ft.

EXCITATION		BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		1*st STORY SHEAR / WEIGHT*		1*st STORY DRIFT / HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
21	WESTERN WASH (325)	7.51	9.83	7.51	9.83	1.00	0.110	0.155	0.110	0.110	0.0594	0.0604
22	EUREKA (022)	6.44	7.70	6.44	7.70	1.00	0.115	0.121	0.073	0.073	0.0396	0.0401
23	EUREKA (023)	16.11	6.43	16.11	6.44	1.00	0.211	0.121	0.107	0.062	0.0579	0.0340
24	FERNDALE (023)	4.53	2.92	4.53	2.92	1.00	0.077	0.073	0.066	0.061	0.0356	0.0335
25	SAN FERNANDO (241)	21.31	13.18	21.31	13.18	1.00	0.273	0.190	0.083	0.052	0.0749	0.0541
26	SAN FERNANDO (458)	14.18	14.16	14.18	14.16	1.00	0.190	0.205	0.095	0.105	0.0518	0.0575
27	SAN FERNANDO (264)	9.71	5.35	9.71	5.35	1.00	0.143	0.104	0.104	0.066	0.0566	0.0362
28	SAN FERNANDO (267)	5.25	6.16	5.25	6.16	1.00	0.102	0.110	0.072	0.071	0.0392	0.0391
29	PUGET SOUND (325)	2.76	10.72	2.76	10.72	1.00	0.073	0.150	0.066	0.095	0.0355	0.0522

	L	T	L	T		L	T	L	T	L	T
MEAN	9.76	8.49	9.76	8.49	1.00	0.144	0.137	0.086	0.077	0.0501	0.0452
σ	5.83	3.53	5.83	3.53	0.00	0.064	0.040	0.017	0.020	0.0127	0.0101
MEAN OF MAX (L,T)	11.14 (8.11) *		11.14 (8.11) *			0.161		0.091		0.0527	
σ OF MAX(L,T)	4.96 (2.20) *		4.96 (2.20) *			0.056		0.016		0.0118	

* Values in parentheses are estimated without the contribution of earthquake excitations No# 23, 25, 26.

TABLE 7.11 Analysis results for 1 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		*1st STORY SHEAR / WEIGHT*		*1st STORY DRIFT / HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
1	HELENA (323)	2.74	0.88	2.74	0.88	1.00	0.125	0.099	0.114	0.084	0.0620	0.0458
2	KERN COUNTY (095)	1.58	1.99	1.58	2.00	1.00	0.113	0.108	0.109	0.116	0.0588	0.0641
3	LYTLE CREEK (290)	1.92	1.41	1.92	1.42	1.00	0.117	0.107	0.099	0.114	0.0538	0.0627
4	PARKFIELD (097)	1.15	0.93	1.15	0.93	1.00	0.109	0.104	0.091	0.133	0.0490	0.0731
5	SAN FERNANDO (284)	1.00	0.98	1.00	0.98	1.00	0.099	0.105	0.189	0.221	0.1021	0.1213
6	SAN FERNANDO (126)	2.35	1.33	2.37	1.35	1.01	0.117	0.111	0.149	0.146	0.0806	0.0797
7	SAN FERNANDO (279)	1.29	1.70	1.29	1.70	1.00	0.105	0.106	0.107	0.138	0.0579	0.0758
8	SAN FERNANDO (104)	1.25	1.51	1.26	1.53	1.01	0.109	0.113	0.161	0.164	0.0871	0.0897
9	SAN FERNANDO (128)	1.59	1.29	1.60	1.30	1.01	0.116	0.113	0.160	0.180	0.0867	0.0985
10	SAN FERNANDO (220)	2.48	0.81	2.48	0.82	1.00	0.125	0.093	0.123	0.132	0.0667	0.0722

MEAN	1.74	1.28	1.74	1.29	1.00	0.113	0.106	0.130	0.143	0.0705	0.0783
σ	0.58	0.37	0.58	0.37	0.00	0.008	0.006	0.031	0.036	0.0166	0.0199
MEAN OF MAX (L,T)	1.84		1.85			0.115		0.146		0.0800	
σ OF MAX(L,T)	0.54		0.053			0.007		0.032		0.0177	

TABLE 7.12 **Analysis results for 1 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.**

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		*1st STORY SHEAR* / *WEIGHT*		*1st STORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
11	LOWER CA (117)	2.99	1.81	3.00	1.82	1.00	0.125	0.111	0.127	0.120	0.0690	0.0660
12	EL CENTRO (117)	2.71	2.31	2.71	2.32	1.00	0.128	0.118	0.102	0.104	0.0554	0.0570
13	PARKFIELD (014)	1.15	1.70	1.16	1.71	1.01	0.107	0.113	0.110	0.120	0.0597	0.0657
14	SAN FERNANDO (110)	4.46	2.39	4.46	2.40	1.00	0.133	0.126	0.135	0.137	0.0731	0.0748
15	SAN FERNANDO (135)	2.18	5.91	2.19	5.92	1.00	0.105	0.155	0.125	0.162	0.0676	0.0887
16	SAN FERNANDO (208)	1.86	2.50	1.86	2.51	1.00	0.115	0.117	0.109	0.108	0.0589	0.0593
17	SAN FERNANDO (211)	1.75	1.90	1.75	1.90	1.00	0.113	0.113	0.107	0.114	0.0581	0.0624
18	SAN FERNANDO (466)	1.00	2.19	1.00	2.20	1.00	0.107	0.118	0.115	0.148	0.0623	0.0813
19	SAN FERNANDO (253)	1.49	2.33	1.49	2.33	1.00	0.109	0.123	0.124	0.148	0.0673	0.0815
20	SAN FERNANDO (199)	2.41	1.67	2.42	1.68	1.00	0.120	0.110	0.129	0.148	0.0698	0.0814

MEAN	2.20	2.47	2.20	2.48	1.00	0.116	0.120	0.118	0.131	0.0641	0.0718
σ	0.97	1.18	0.97	1.18	0.00	0.009	0.013	0.010	0.019	0.0057	0.0105
MEAN OF MAX (L,T)	2.91		2.92			0.125		0.132		0.0721	
σ OF MAX(L,T)	1.23		1.23			0.012		0.019		0.0104	

TABLE 7.13 Analysis results for 1 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Weight is total weight of structure including base. Height is 12ft:

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	BASE SHEAR / WEIGHT		1st STORY SHEAR / WEIGHT		1st STORY DRIFT / HEIGHT (%)	
		L	T	L	T		L	T	L	T	L	T
21	WESTERN WASH (325)	4.10	2.91	4.11	2.93	1.01	0.132	0.125	0.152	0.153	0.0826	0.0840
22	EUREKA (022)	3.69	3.78	3.69	3.78	1.00	0.126	0.125	0.080	0.112	0.0433	0.0614
23	EUREKA (023)	11.76	5.72	11.76	5.73	1.00	0.200	0.161	0.103	0.087	0.0559	0.0474
24	FERNDALE (023)	4.64	3.58	4.64	3.58	1.00	0.122	0.133	0.109	0.100	0.0590	0.0548
25	SAN FERNANDO (241)	4.22	5.54	4.22	5.54	1.00	0.131	0.160	0.100	0.125	0.0541	0.0685
26	SAN FERNANDO (458)	5.40	4.52	5.41	4.52	1.00	0.152	0.146	0.118	0.091	0.0638	0.0498
27	SAN FERNANDO (264)	6.57	3.37	6.58	3.37	1.00	0.164	0.123	0.145	0.120	0.0785	0.0661
28	SAN FERNANDO (267)	4.10	3.95	4.10	3.95	1.00	0.144	0.138	0.131	0.175	0.0710	0.0963
29	PUGET SOUND (325)	2.92	6.15	2.92	6.16	1.00	0.122	0.168	0.125	0.168	0.0679	0.0917

MEAN	5.27	4.39	5.27	4.40	1.00	0.144	0.142	0.118	0.126	0.0640	0.0689
σ	2.50	1.09	2.50	1.09	0.00	0.024	0.016	0.022	0.031	0.0117	0.0170
MEAN OF MAX (L,T)	5.78 (5.04) *		5.78 (5.04) *			0.153		0.134		0.0732	
σ OF MAX(L,T)	2.30 (0.97) *		2.30 (0.97) *			0.022		0.025		0.0140	

* Values in parentheses are estimated without the contribution of earthquake excitation No# 23.

TABLE 7.14 **Analysis results for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.**

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		*1stSTORY SHEAR / WEIGHT*		*1stSTORY DRIFT / HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
1	HELENA (323)	3.64	0.89	3.72	1.05	1.02	0.188	0.069	0.173	0.067	0.4093	0.1613
2	KERN COUNTY (095)	1.78	1.18	1.79	1.25	1.01	0.123	0.121	0.119	0.112	0.2800	0.2750
3	LYTLE CREEK (290)	0.88	0.50	0.89	0.52	1.01	0.113	0.102	0.113	0.118	0.2730	0.2717
4	PARKFIELD (097)	0.81	1.16	0.82	1.27	1.09	0.103	0.116	0.093	0.112	0.2165	0.2701
5	SAN FERNANDO (284)	1.12	1.34	1.17	1.44	1.07	0.108	0.116	0.109	0.115	0.2567	0.2713
6	SAN FERNANDO (126)	1.62	0.88	1.62	0.90	1.00	0.122	0.102	0.127	0.099	0.2974	0.2368
7	SAN FERNANDO (279)	1.80	2.87	1.85	2.97	1.03	0.130	0.143	0.115	0.130	0.2698	0.3097
8	SAN FERNANDO (104)	0.49	0.62	0.52	0.70	1.13	0.096	0.105	0.090	0.105	0.2121	0.2533
9	SAN FERNANDO (128)	0.49	0.68	0.50	0.71	1.04	0.099	0.111	0.129	0.127	0.3062	0.3045
10	SAN FERNANDO (220)	1.33	0.97	1.37	1.05	1.03	0.123	0.104	0.112	0.101	0.2659	0.2381

MEAN	1.40	1.11	1.43	1.19	1.04	0.121	0.109	0.118	0.109	0.2787	0.2592
σ	0.88	0.64	0.90	0.65	0.04	0.025	0.018	0.022	0.017	0.0522	0.0398
MEAN OF MAX (L,T)	1.59		1.65			0.126		0.124		0.2936	
σ OF MAX(L,T)	0.92		0.93			0.023		0.018		0.0422	

TABLE 7.15 Analysis results for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	$\frac{\textit{BASE SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{1stSTORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{1stSTORY DRIFT}}{\textit{HEIGHT}}$ (%)	
		L	T	L	T		L	T	L	T	L	T
11	LOWER CA (117)	2.67	0.97	2.69	0.99	1.01	0.152	0.114	0.143	0.124	0.3350	0.2988
12	EL CENTRO (117)	3.05	2.90	3.21	3.09	1.05	0.156	0.146	0.153	0.136	0.3554	0.3271
13	PARKFIELD (014)	0.78	1.72	0.81	1.92	1.12	0.117	0.128	0.100	0.128	0.2370	0.3005
14	SAN FERNANDO (110)	3.14	1.51	3.16	1.57	1.01	0.154	0.123	0.149	0.116	0.3458	0.2750
15	SAN FERNANDO (135)	1.98	4.77	2.00	4.81	1.01	0.103	0.209	0.101	0.182	0.2419	0.4267
16	SAN FERNANDO (208)	1.83	1.96	1.90	2.11	1.08	0.133	0.125	0.121	0.119	0.2869	0.2832
17	SAN FERNANDO (211)	2.13	1.66	2.17	1.72	1.02	0.131	0.126	0.126	0.123	0.3010	0.2888
18	SAN FERNANDO (466)	4.55	4.33	4.70	4.61	1.03	0.202	0.193	0.187	0.166	0.4313	0.3899
19	SAN FERNANDO (253)	1.23	2.36	1.32	2.58	1.09	0.120	0.142	0.110	0.136	0.2593	0.3197
20	SAN FERNANDO (199)	1.84	2.48	1.88	2.53	1.02	0.132	0.129	0.124	0.122	0.2935	0.2890

MEAN	2.32	2.47	2.38	2.59	1.04	0.140	0.144	0.131	0.135	0.3087	0.3199
σ	1.02	1.17	1.05	1.19	0.04	0.026	0.030	0.026	0.021	0.0566	0.0474
MEAN OF MAX (L,T)	2.88		2.99			0.154		0.145		0.3396	
σ OF MAX(L,T)	0.98		0.97			0.026		0.022		0.0496	

TABLE 7.16 Analysis results for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Weight is total weight of structure including base. Height is 12ft.

EXCITATION		BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		*1stSTORY SHEAR / WEIGHT*		*1stSTORY DRIFT / HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
21	WESTERN WASH (325)	2.99	3.51	3.10	3.73	1.06	0.162	0.181	0.169	0.180	0.4099	0.4333
22	EUREKA (022)	3.55	3.25	3.57	3.29	1.01	0.150	0.159	0.134	0.174	0.3134	0.4213
23	EUREKA (023)	10.50	5.53	10.58	5.70	1.01	0.324	0.231	0.308	0.219	0.7159	0.5089
24	FERNDALE (023)	2.29	1.16	2.40	1.23	1.05	0.125	0.128	0.132	0.127	0.3057	0.3125
25	SAN FERNANDO (241)	7.27	7.02	7.35	7.11	1.01	0.264	0.248	0.237	0.227	0.5540	0.5278
26	SAN FERNANDO (458)	6.71	4.89	6.77	5.07	1.01	0.263	0.218	0.232	0.193	0.5319	0.4465
27	SAN FERNANDO (264)	6.46	3.41	6.54	3.55	1.01	0.250	0.145	0.241	0.135	0.5678	0.3207
28	SAN FERNANDO (267)	4.16	3.58	4.18	3.59	1.00	0.195	0.172	0.179	0.181	0.4231	0.4351
29	PUGET SOUND (325)	2.64	7.83	2.78	7.85	1.00	0.145	0.293	0.135	0.273	0.3215	0.6451

MEAN	5.17	4.46	5.25	4.57	1.02	0.209	0.197	0.196	0.190	0.4604	0.4501
σ	2.58	1.95	2.57	1.95	0.02	0.065	0.051	0.058	0.043	0.1331	0.0969
MEAN OF MAX (L,T)	5.81 (5.22) *		5.89 (5.30) *			0.229		0.218		0.5130	
σ OF MAX(L,T)	2.47 (1.94) *		2.46 (1.92) *			0.062		0.052		0.1177	

* Values in parentheses are estimated without the contribution of earthquake excitation No# 23.

TABLE 7.17 Analysis results for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		*1stSTORY SHEAR* / *WEIGHT*		*1stSTORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
1	HELENA (323)	3.63	1.42	3.68	1.50	1.01	0.087	0.042	0.082	0.038	0.1939	0.0926
2	KERN COUNTY (095)	1.92	2.41	1.93	2.43	1.01	0.061	0.073	0.060	0.071	0.1463	0.1664
3	LYTLE CREEK (290)	0.81	1.00	0.84	1.05	1.05	0.057	0.057	0.077	0.068	0.1834	0.1565
4	PARKFIELD (097)	1.27	1.52	1.30	1.53	1.01	0.063	0.064	0.056	0.067	0.1292	0.1603
5	SAN FERNANDO (284)	1.86	2.77	1.89	2.82	1.02	0.058	0.080	0.058	0.069	0.1400	0.1666
6	SAN FERNANDO (126)	2.41	1.16	2.45	1.24	1.02	0.071	0.056	0.076	0.055	0.1861	0.1336
7	SAN FERNANDO (279)	2.32	3.93	2.36	3.99	1.02	0.074	0.077	0.063	0.070	0.1466	0.1680
8	SAN FERNANDO (104)	1.11	1.54	1.16	1.55	1.01	0.057	0.063	0.055	0.067	0.1327	0.1655
9	SAN FERNANDO (128)	1.37	1.51	1.40	1.53	1.01	0.061	0.064	0.067	0.072	0.1562	0.1771
10	SAN FERNANDO (220)	2.67	1.67	2.71	1.73	1.01	0.065	0.064	0.070	0.066	0.1643	0.1585

MEAN	1.94	1.89	1.97	1.94	1.02	0.065	0.064	0.066	0.064	0.1579	0.1545
σ	0.81	0.85	0.81	0.85	0.01	0.009	0.011	0.009	0.010	0.0220	0.0233
MEAN OF MAX (L,T)	2.34		2.37			0.070		0.072		0.01732	
σ OF MAX(L,T)	0.91		0.92			0.009		0.005		0.00107	

TABLE 7.18 Analysis results for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.

EXCITATION		BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		*1stSTORY SHEAR* / *WEIGHT*		*1stSTORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
11	LOWER CA (117)	3.91	2.11	3.96	2.13	1.01	0.083	0.064	0.075	0.076	0.1756	0.1870
12	EL CENTRO (117)	7.98	5.78	8.09	5.89	1.01	0.139	0.099	0.119	0.089	0.2779	0.2159
13	PARKFIELD (014)	1.16	2.14	1.16	2.17	1.01	0.060	0.068	0.054	0.082	0.1251	0.1952
14	SAN FERNANDO (110)	5.31	2.75	5.34	2.80	1.01	0.096	0.067	0.093	0.070	0.2175	0.1631
15	SAN FERNANDO (135)	4.30	12.17	4.35	12.20	1.00	0.081	0.184	0.092	0.167	0.2174	0.3933
16	SAN FERNANDO (208)	5.22	6.50	5.24	6.57	1.01	0.093	0.107	0.083	0.101	0.1960	0.2352
17	SAN FERNANDO (211)	6.34	5.99	6.34	6.07	1.00	0.110	0.096	0.099	0.095	0.2324	0.2227
18	SAN FERNANDO (466)	6.27	12.27	6.38	12.37	1.01	0.118	0.171	0.118	0.158	0.2747	0.3718
19	SAN FERNANDO (253)	6.28	13.04	6.35	13.14	1.01	0.082	0.190	0.073	0.172	0.1695	0.4066
20	SAN FERNANDO (199)	4.11	2.58	4.12	2.67	1.00	0.078	0.075	0.081	0.079	0.1979	0.1887

MEAN	5.10	6.53	5.13	6.60	1.01	0.094	0.112	0.089	0.109	0.2084	0.2580
σ	1.77	4.20	1.79	4.22	0.00	0.022	0.048	0.019	0.038	0.0444	0.0893
MEAN OF MAX (L,T)	7.38 (5.18) *		7.43 (5.23) *			0.123		0.115		0.2715	
σ OF MAX(L,T)	3.68 (1.81) *		3.70 (1.83) *			0.043		0.035		0.0821	

* Values in parentheses are estimated without the contribution of earthquake excitations No# 15, 18, 19.

TABLE 7.19 Analysis results for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Weight is total weight of structure including base. Height is 12ft.

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	$\frac{BASE\ SHEAR}{WEIGHT}$		$\frac{1stSTORY\ SHEAR}{WEIGHT}$		$\frac{1stSTORY\ DRIFT}{HEIGHT}$ (%)	
		L	T	L	T		L	T	L	T	L	T
21	WESTERN WASH (325)	7.36	10.97	7.43	11.17	1.02	0.102	0.163	0.105	0.139	0.2459	0.3261
22	EUREKA (022)	6.96	7.96	7.04	8.00	1.01	0.112	0.117	0.105	0.109	0.2448	0.2578
23	EUREKA (023)	14.73	5.58	14.76	5.66	1.00	0.197	0.106	0.185	0.093	0.4317	0.2146
24	FERNDALE (023)	2.89	1.99	2.93	2.05	1.01	0.062	0.072	0.069	0.071	0.1599	0.1740
25	SAN FERNANDO (241)	19.32	12.03	19.37	12.12	1.00	0.267	0.182	0.233	0.163	0.5415	0.3812
26	SAN FERNANDO (458)	15.33	14.71	15.36	14.82	1.00	0.208	0.209	0.185	0.188	0.4295	0.4391
27	SAN FERNANDO (264)	8.08	5.45	8.12	5.46	1.00	0.124	0.108	0.114	0.094	0.2662	0.2174
28	SAN FERNANDO (267)	5.34	6.26	5.35	6.39	1.02	0.098	0.109	0.091	0.104	0.2137	0.2496
29	PUGET SOUND (325)	2.97	8.93	3.08	9.06	1.01	0.063	0.145	0.071	0.134	0.1657	0.3116

MEAN	9.22	8.21	9.27	8.30	1.01	0.137	0.135	0.129	0.122	0.2999	0.2857
σ	5.51	3.68	5.50	3.71	0.01	0.067	0.041	0.055	0.035	0.1268	0.0811
MEAN OF MAX (L,T)	10.50 (7.52) *		10.57 (7.62) *			0.156		0.142		0.3331	
σ OF MAX(L,T)	4.83 (2.50) *		4.82 (2.54) *			0.057		0.048		0.1092	

* Values in parentheses are estimated without the contribution of earthquake excitations No# 23, 25, 26.

TABLE 7.20 Analysis results for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		1st*STORY SHEAR / WEIGHT*		1st*STORY DRIFT / HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
1	HELENA (323)	4.30	1.05	4.39	1.21	1.02	0.143	0.061	0.135	0.060	0.3192	0.1409
2	KERN COUNTY (095)	1.94	1.18	1.96	1.23	1.01	0.104	0.104	0.102	0.100	0.2407	0.2482
3	LYTLE CREEK (290)	0.98	0.62	0.98	0.63	1.00	0.104	0.099	0.113	0.117	0.2733	0.2690
4	PARKFIELD (097)	0.95	1.08	0.98	1.19	1.10	0.098	0.105	0.092	0.123	0.2128	0.2886
5	SAN FERNANDO (284)	1.36	1.27	1.45	1.48	1.17	0.104	0.105	0.105	0.115	0.2481	0.2737
6	SAN FERNANDO (126)	1.59	0.81	1.60	0.84	1.01	0.101	0.090	0.130	0.098	0.3055	0.2362
7	SAN FERNANDO (279)	1.57	2.48	1.59	2.53	1.01	0.106	0.104	0.102	0.099	0.2396	0.2364
8	SAN FERNANDO (104)	0.57	0.65	0.59	0.76	1.17	0.094	0.100	0.088	0.102	0.2114	0.2446
9	SAN FERNANDO (128)	0.53	0.91	0.55	0.93	1.02	0.097	0.106	0.132	0.123	0.3127	0.2938
10	SAN FERNANDO (220)	1.62	1.13	1.69	1.26	1.04	0.107	0.098	0.109	0.099	0.2591	0.2327

MEAN	1.54	1.12	1.58	1.21	1.06	0.106	0.097	0.111	0.104	0.2622	0.2464
σ	1.02	0.50	1.04	0.51	0.06	0.013	0.013	0.016	0.017	0.0374	0.0410
MEAN OF MAX (L,T)	1.69		1.75			0.108		0.117		0.2760	
σ OF MAX(L,T)	1.01		1.01			0.012		0.012		0.0277	

TABLE 7.21 Analysis results for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Weight is total weight of structure including base. Height is 12ft.

	EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		1*stSTORY SHEAR* / *WEIGHT*		1*stSTORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T		L	T	L	T	L	T
11	LOWER CA (117)	3.11	1.06	3.15	1.12	1.01	0.118	0.107	0.118	0.125	0.2782	0.3020
12	EL CENTRO (117)	3.95	3.48	4.07	3.54	1.03	0.138	0.131	0.122	0.122	0.2828	0.2941
13	PARKFIELD (014)	0.66	1.63	0.69	1.83	1.12	0.105	0.110	0.099	0.137	0.2352	0.3208
14	SAN FERNANDO (110)	3.82	1.90	3.85	1.96	1.03	0.122	0.099	0.121	0.110	0.2890	0.2637
15	SAN FERNANDO (135)	2.36	5.62	2.37	5.73	1.02	0.095	0.154	0.105	0.143	0.2477	0.3359
16	SAN FERNANDO (208)	1.99	2.33	2.14	2.43	1.04	0.118	0.113	0.108	0.118	0.2552	0.2828
17	SAN FERNANDO (211)	2.35	2.25	2.45	2.35	1.04	0.114	0.117	0.117	0.118	0.2754	0.2821
18	SAN FERNANDO (466)	5.05	4.70	5.07	4.86	1.00	0.153	0.148	0.125	0.129	0.2872	0.3082
19	SAN FERNANDO (253)	1.81	2.88	1.88	3.10	1.08	0.110	0.119	0.105	0.126	0.2497	0.3023
20	SAN FERNANDO (199)	2.15	2.50	2.21	2.56	1.02	0.107	0.111	0.121	0.110	0.2887	0.2617

MEAN	2.73	2.84	2.79	2.71	1.04	0.118	0.121	0.114	0.124	0.2689	0.2954
σ	1.20	1.34	1.20	1.41	0.03	0.016	0.017	0.009	0.010	0.0190	0.0224
MEAN OF MAX (L,T)	3.32		3.42			0.126		0.126		0.3006	
σ OF MAX(L,T)	1.21		1.18			0.016		0.008		0.00164	

TABLE 7.22 Analysis results for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Weight is total weight of structure including base. Height is 12ft.

EXCITATION		BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	$\frac{\textit{BASE SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{1stSTORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{1stSTORY DRIFT}}{\textit{HEIGHT}}$ (%)	
		L	T	L	T		L	T	L	T	L	T
21	WESTERN WASH (325)	3.64	4.40	3.68	4.69	1.07	0.123	0.145	0.137	0.142	0.3358	0.3449
22	EUREKA (022)	4.53	4.03	4.56	4.06	1.01	0.127	0.129	0.124	0.148	0.2835	0.3620
23	EUREKA (023)	12.34	5.01	12.39	5.12	1.00	0.193	0.150	0.171	0.128	0.3930	0.2963
24	FERNDALE (023)	2.48	1.32	2.57	1.39	1.04	0.105	0.114	0.116	0.116	0.2730	0.2871
25	SAN FERNANDO (241)	8.17	7.51	8.27	7.58	1.01	0.153	0.171	0.141	0.151	0.3335	0.3519
26	SAN FERNANDO (458)	5.78	6.73	5.97	6.93	1.03	0.155	0.167	0.130	0.163	0.2963	0.3822
27	SAN FERNANDO (264)	6.37	3.58	6.38	3.69	1.00	0.156	0.117	0.161	0.127	0.3848	0.3100
28	SAN FERNANDO (267)	5.13	4.20	5.18	4.28	1.01	0.145	0.135	0.130	0.158	0.3080	0.3843
29	PUGET SOUND (325)	3.60	6.46	3.68	6.51	1.01	0.102	0.168	0.117	0.165	0.2742	0.4007

MEAN	5.78	4.80	5.85	4.92	1.02	0.140	0.144	0.136	0.144	0.3202	0.3466
σ	2.81	1.78	2.81	1.79	0.02	0.027	0.020	0.018	0.016	0.0426	0.0384
MEAN OF MAX (L,T)	6.29 (5.54) *		6.39 (5.64) *			0.154		0.153		0.3657	
σ OF MAX(L,T)	2.64 (1.65) *		2.63 (1.64) *			0.023		0.016		0.00329	

TABLE 7.23 Summary of results of the mean and the mean plus one standard deviation values of the base displacement for 1 - story isolated structure excited by PGV scaled earthquake time histories and comparison of these results with design values according to SEAOC static analysis procedure. Units are inches.

Soil Type	Sliding Isolation System Properties								
	R=39.132 in (T_b = 2 sec.) fmax=0.10			R=88.048 in (T_b = 3 sec.) fmax=0.05			R=88.048 in (T_b = 3 sec.) fmax=0.10		
	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC
S1 Rock Sites	1.77	2.23	2.81	2.72	3.54	5.28	1.84	2.38	3.11
S1 Stiff Soil Sites	2.52	3.59	2.81	6.90	10.10	5.28	2.91	4.14	3.11
S2 Medium Soil Sites	5.94	8.53	5.72	11.14	16.10	10.19	5.78	8.08	6.32

TABLE 7.24 Summary of results of the mean and the mean plus one standard deviation values of base displacement for 8 - story isolated structure excited by PGV scaled earthquake time histories and comparison of these results with the design values according to SEAOC static analysis procedure. Units are inches.

Soil Type	Sliding Isolation System Properties								
	R=39.132 in (T_b = 2 sec.) fmax=0.10			R=88.048 in (T_b = 3 sec.) fmax=0.05			R=88.048 in (T_b = 3 sec.) fmax=0.10		
	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC
S1 Rock Sites	1.59	2.51	2.81	2.34	3.25	5.28	1.69	2.70	3.11
S1 Stiff Soil Sites	2.88	3.86	2.81	7.38	11.06	5.28	3.32	4.53	3.11
S2 Medium Soil Sites	5.81	8.28	5.72	10.50	15.33	10.19	6.29	8.93	6.32

TABLE 7.25 Summary of results of the mean and the mean plus one standard deviation values of the base displacement for 1 - story isolated structure excited by PGV scaled earthquake time histories considered in this study representative of the soil sites they were recorded and comparison of these results with the design values according to SEAOC static analysis procedure. Units are inches. Certain records were removed as indicated in Tables (7.5) to (7.22).

Soil Type	Sliding Isolation System Properties								
	R=39.132 in (T_b = 2 sec.) fmax=0.10			R=88.048 in (T_b = 3 sec.) fmax=0.05			R=88.048 in (T_b = 3 sec.) fmax=0.10		
	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC
S1 Rock Sites	1.77	2.23	2.81	2.72	3.54	5.28	1.84	2.38	3.11
S1 Stiff Soil Sites	2.52	3.59	2.81	4.98	6.52	5.28	2.91	4.14	3.11
S2 Medium Soil Sites	5.25	7.07	5.72	8.11	10.31	10.19	5.04	6.01	6.32

TABLE 7.26 Summary of results of the mean and the mean plus one standard deviation values of the base displacement for 8 - story isolated structure excited by PGV scaled earthquake time histories considered in this study representative of the soil sites they were recorded and comparison of these results with the design values according to SEAOC static analysis procedure. Units are inches. Certain records were removed as indicated in Tables (7.5) to (7.22).

Soil Type	Sliding Isolation System Properties								
	R=39.132 in (T_b = 2 sec.) fmax=0.10			R=88.048 in (T_b = 3 sec.) fmax=0.05			R=88.048 in (T_b = 3 sec.) fmax=0.10		
	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC	Mean of max (L,T)	Mean plus 1σ of max (L,T)	SEAOC
S1 Rock Sites	1.59	2.51	2.81	2.34	3.25	5.28	1.69	2.70	3.11
S1 Stiff Soil Sites	2.88	3.86	2.81	5.18	6.99	5.28	3.32	4.53	3.11
S2 Medium Soil Sites	5.22	7.16	5.72	7.52	10.02	10.19	5.54	7.19	6.32

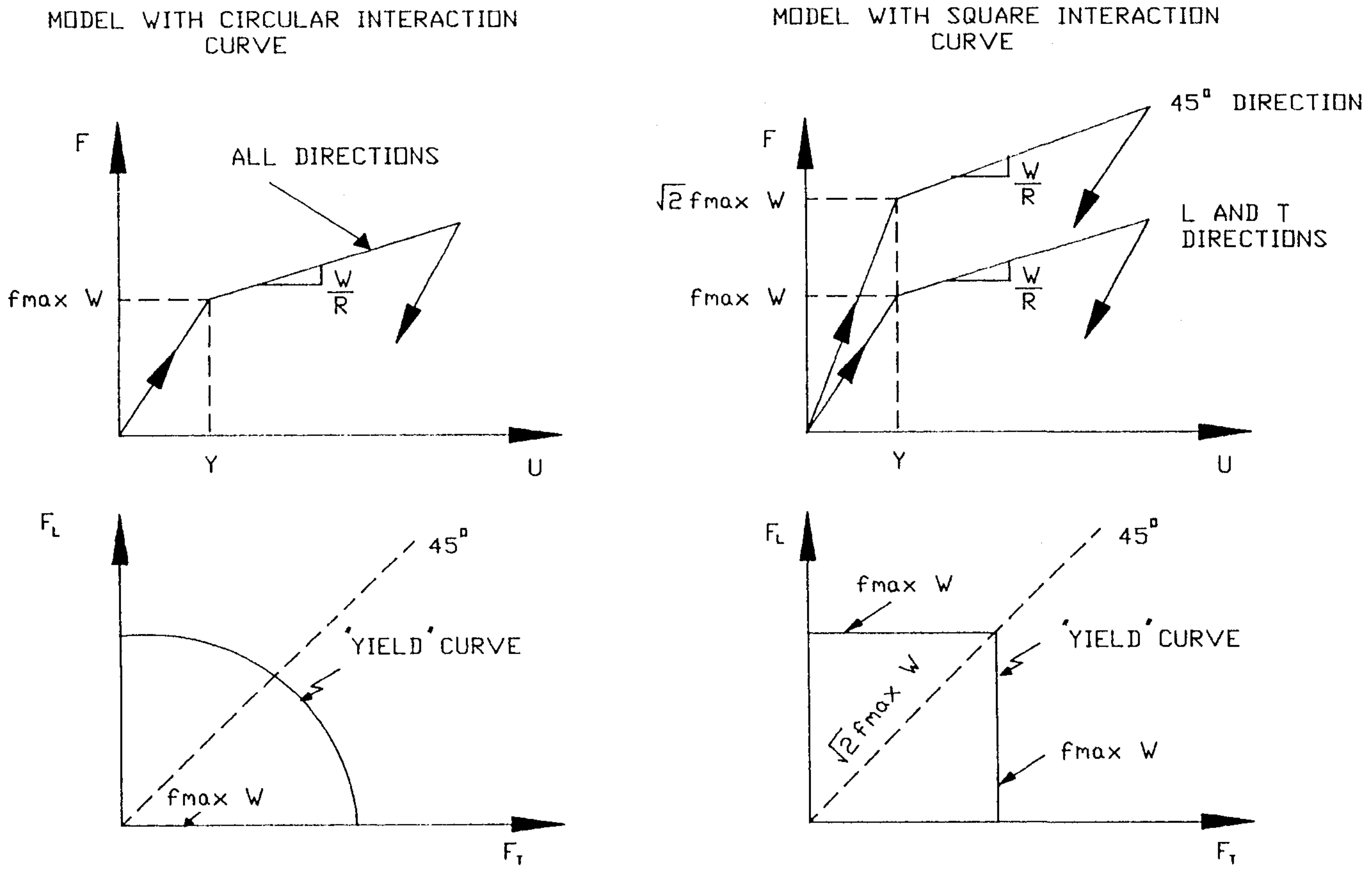


FIGURE 7-4 Comparison of circular and square interaction models of isolation bearings.

TABLE 7.27 Analysis results for 1-story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on medium soil sites (representative of soil type S2). Weight is total weight of structure including base. Height is 12 ft. Square interaction model for isolation bearings.

EXCITATION		BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	BASE SHEAR / WEIGHT		1ST STORY SHEAR / WEIGHT		1ST STORY DRIFT / HEIGHT %	
		L	T	L	T		L	T	L	T	L	T
21	WESTERN WASH (325)	3.29	3.07	3.30	3.08	1.00	0.181	0.175	0.166	0.213	0.0898	0.1181
22	EUREKA (022)	2.40	2.37	2.40	2.39	1.00	0.157	0.157	0.121	0.163	0.0682	0.0901
23	EUREKA (023)	8.80	4.80	8.80	4.80	1.00	0.323	0.220	0.170	0.146	0.0924	0.0804
24	FERNDALE (023)	4.18	3.05	4.19	3.05	1.00	0.205	0.175	0.160	0.129	0.0869	0.0713
25	SAN FERNANDO (241)	2.08	5.43	2.09	5.44	1.00	0.147	0.237	0.139	0.161	0.0756	0.0893
26	SAN FERNANDO (458)	3.32	2.28	3.32	2.29	1.00	0.181	0.153	0.123	0.119	0.0667	0.0657
27	SAN FERNANDO (264)	6.27	2.65	6.27	2.67	1.00	0.256	0.166	0.172	0.149	0.0934	0.0827
28	SAN FERNANDO (267)	3.86	3.82	3.86	3.82	1.00	0.196	0.196	0.160	0.160	0.0869	0.0886
29	PUGET SOUND (325)	1.87	6.96	1.89	6.96	1.00	0.170	0.224	0.146	0.276	0.0921	0.1235

MEAN	4.00	3.83	4.01	3.83	1.00	0.202	0.189	0.151	0.168	0.0836	0.0900
σ	2.11	1.51	2.11	1.51	0.00	0.052	0.029	0.018	0.045	0.0099	0.0183
MEAN OF MAX (L,T)	4.95		4.95			0.218		0.178		0.0943	
σ OF MAX(L,T)	1.96		1.96			0.047		0.041		0.0160	

SECTION 8

TIME HISTORY ANALYSES ON THE BASIS OF SEAOC DYNAMIC ANALYSIS REQUIREMENTS

In this section, a different approach is used for the evaluation of the code requirements related to seismic isolation. The methodology that follows is based on the SEAOC specifications for time history analysis. Thus, comparisons can be made between the results of the dynamic analysis approach according to the SEAOC specifications and the static analysis procedure, as prescribed by the design formula of SEAOC.

According to SEAOC requirements, referring to time history analysis, pairs of horizontal ground motion time history components shall be selected from at least three recorded events. These motions shall be scaled appropriately, so that the square root of the sum of the squares (SRSS) of the 5% damped spectrum of the horizontal scaled components does not fall below 1.3 times the 5% damped spectrum of the Design-Basis Earthquake by more than 10% in the period range of T minus 1.0 seconds to T plus 2.0 seconds, where T is the period as determined by equation 2.4.

In order to comply with the above requirements, 6 pairs of earthquake records were selected from the total of 29 that were used in the series of statistical evaluation analyses, as discussed in Section 7. Three of those records were recorded on rock or stiff soil sites (soil type S1) and three of them on medium soil sites (soil type S2). The horizontal components of each record were scaled in amplitude separately, by contrast with the methodology followed in the series of the statistical evaluation analyses, (i.e. scaling both components by a common factor). The scaling factors were estimated appropriately after performing trials, so that the square root of the sum of the squares of the 5% damped spectrum of the scaled horizontal components will be as consistent as possible with the desired one, according to the SEAOC specifications. No time scaling of the records was employed.

This scaling approach was followed by an effort to make the SRSS spectrum of the scaled horizontal components have a lower bound not more than 10% less of the 1.3 times of the 5% damped spectrum of the Design-Basis Earthquake, as specified by SEAOC, and also be smooth and comparable in shape with the one of the Design-Basis Earthquake.

In fact, almost all of the 5% damping spectra of the horizontal components of the selected earthquake records included certain peaks in the period range under consideration, and they were not attenuating smoothly as the period increased the way that the design spectrum does. Thus, an effort was made as to predict the best combination of scaling factors, so that the shape of the SRSS spectra approximated the Design-Basis spectra. The lower bound limit criteria for the SRSS 5% damped spectra of the selected motions were satisfied almost everywhere in the period range under consideration (0.5 seconds to 3.5 seconds). Furthermore, the average of the three SRSS spectra in each case of soil type was above the 1.3 times the Design-Basis spectrum.

The 5% damping spectra of the selected earthquake motions and their respective SRSS spectra are shown in Figures 8-1 to 8-2. Table 8.1 is a list of the motions selected, the factors that scaled their components and the resulting PGA and PGV values of those components. One can observe that the resulted peak ground accelerations are generally higher than 0.4g, having a maximum value of 1.313g. Likewise, the peak ground velocity values had a maximum of 23.66 in/sec for S1 soil types and 27.37 in/sec for S2 soil types. Those values can be compared to the values that were determined to be the basis for scaling according to PGA (PGA = 0.4g) or PGV (PGV = 12in/sec for rock sites, 18in/sec for stiff soil sites and 22.5 in/sec for medium soil sites) applied in the series of the statistical evaluation analyses.

Finally, it should be noted that in all the selected motions, the component that yielded the most intense 5% damping spectrum was the one applied in the transverse (T) direction of the structure. In this way, the strongest earthquake component was coupled with the mass eccentricity to create torsional motion.

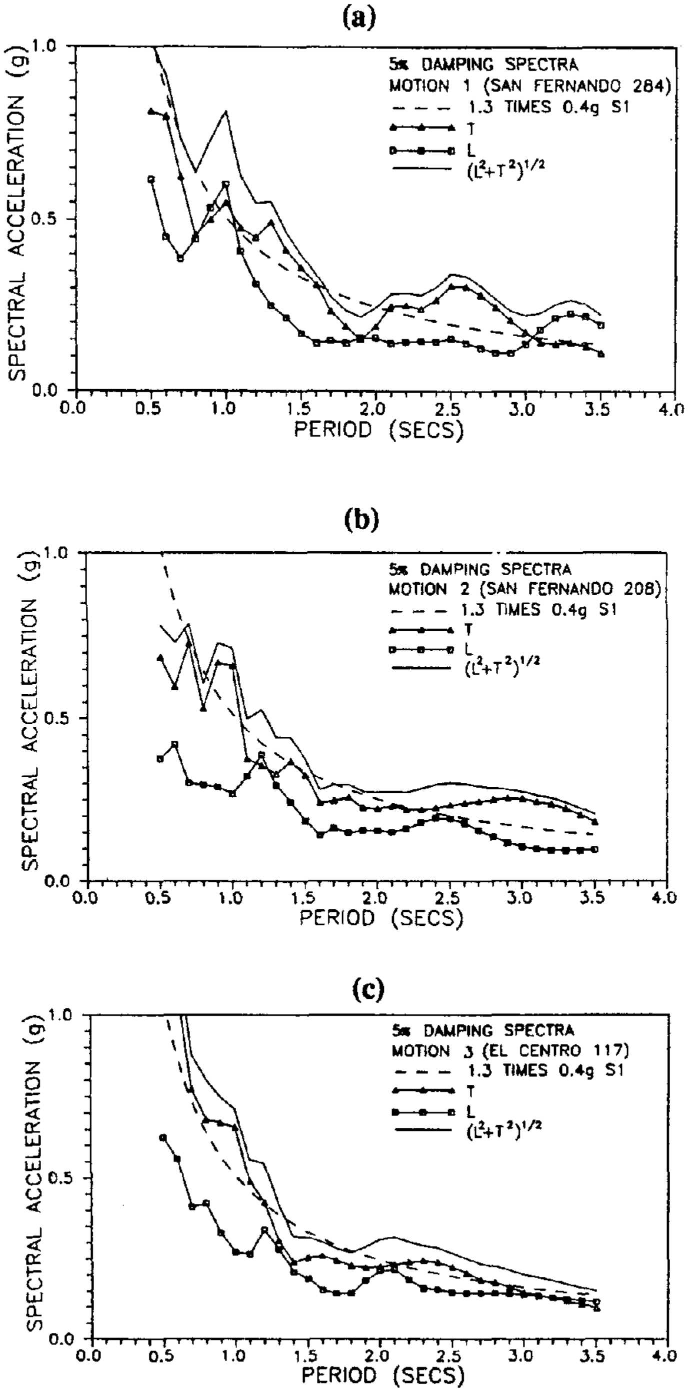


FIGURE 8-1 5% damping spectra of components T,L and square root of sum of squares of components T and L of scaled earthquake motions selected for dynamic analyses according to SEAOC time history analysis procedure and comparison with 1.3 times 0.4g S1 Design Spectrum.

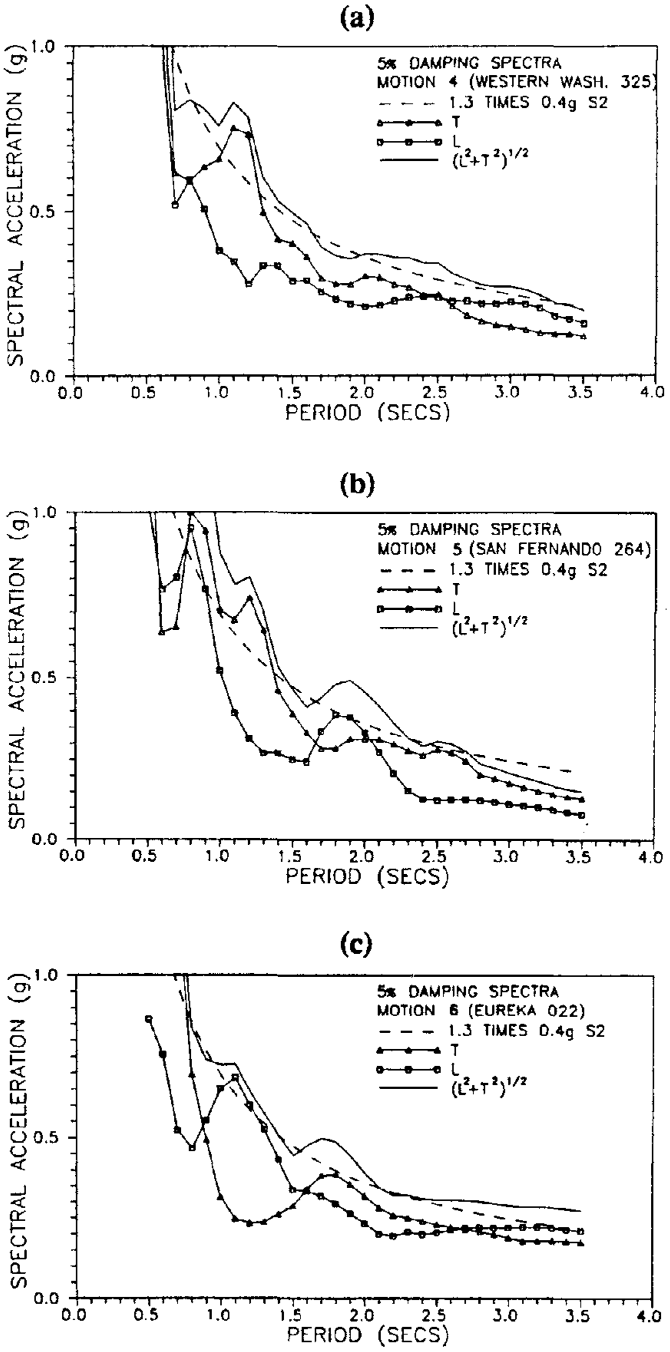


FIGURE 8-2 5% damping spectra of components T,L and square root of sum of squares of components T and L of scaled earthquake motions selected for dynamic analyses according to SEAOC time history analysis procedure and comparison with 1.3 times 0.4g S2 Design Spectrum.

TABLE 8.1 Selected time histories and their scaling factors.

EXCITATION	SOIL TYPE	ORIGINAL RECORD	COMPONENTS		SC. FACTOR		PGA (g)		PGV (in/sec).	
			T	L	T	L	T	L	T	L
MOTION #1	S1	SAN FERNANDO (284)	S82W	S08E	6.50	3.70	1.313	0.803	15.990	14.393
MOTION #2	S1	SAN FERNANDO (208)	N00E	S90W	2.40	1.80	0.326	0.259	21.096	13.140
MOTION #3	S1	EL CENTRO (117)	S00E	S90W	1.30	1.00	0.452	0.214	17.121	14.530
MOTION #4	S2	WESTERN WASH (325)	N04W	N86E	2.98	2.10	0.492	0.588	25.121	14.112
MOTION #5	S2	SAN FERNANDO (264)	N90E	N00E	3.85	3.85	0.712	0.774	24.910	14.900
MOTION #6	S2	EUREKA (022)	N79E	N11W	2.35	2.20	0.606	0.370	27.166	27.368

8.1 Comparison of Time History Analysis Results to SEAOC Design Formulae.

The analysis results are presented in Tables 8.2 to 8.7. Additional results for the story shear and the interstory drift ratios are presented in Appendix D. These results show that the corner to center ratio of the isolation system displacement is equal to unity in the case of the 1 - story structure. Furthermore, this ratio is larger than unity in the 8 - story structure. These results are consistent with those obtained in the dynamic analyses of Section 7.

Tables 8.8 and 8.9 compare the SEAOC design procedure to the results of the time history analysis. The tables include the maximum displacement at the geometric center of the base (maximum among T and L components of all three records) and the value of the displacement according to the SEAOC static procedure. It may be observed that for soil type S2, the analysis results are in good agreement with the SEAOC displacement. The SEAOC formula predicts displacements with error of less than 10% of the calculated value. In the case of soil type S1, the SEAOC formula underestimates the calculated displacement (by as much as 25%) for the system with $T_b = 3$ seconds. For this system, the effective period T (equation 2.4) is about 2.5 seconds. From the spectra of Figure 8-1, it can be observed that at this period, the spectra for motions of soil type S1 overestimate the target spectrum by as much as 30%. This should explain the difference.

It may be concluded that the procedure employed in this section produces results that are in agreement with those obtained in Section 7. Both procedures (the one based on statistical evaluation of the response and the one based on scaled records according to the dynamic analysis procedure of SEAOC) are consistent with the results of SEAOC static analysis procedure.

TABLE 8.2 Analysis results for 1 - story isolated structure with R=39.132 in (T_b = 2 sec), f_{max} = 0.10, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Weight is total weight of structure including base. Height is 12ft.

EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		1*st STORY SHEAR / WEIGHT*		1*st STORY DRIFT / HEIGHT* (%)	
	L	T	L	T		L	T	L	T	L	T
MOTION #1	1.01	1.78	1.02	1.78	1.00	0.106	0.143	0.191	0.223	0.1038	0.1225
MOTION #2	0.99	2.11	0.99	2.11	1.00	0.098	0.139	0.102	0.129	0.0551	0.0707
MOTION #3	1.50	2.16	1.51	2.16	1.00	0.135	0.143	0.084	0.119	0.0453	0.0652
MOTION #4	1.97	3.31	1.97	3.31	1.00	0.133	0.171	0.118	0.155	0.0643	0.0851
MOTION #5	2.17	5.69	2.17	5.69	1.00	0.116	0.210	0.115	0.142	0.0627	0.0782
MOTION #6	3.75	4.52	3.75	4.52	1.00	0.165	0.202	0.123	0.136	0.0670	0.0745

TABLE 8.3 Analysis results for 1 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.05, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Weight is total weight of structure including base. Height is 12ft.

EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		1*st STORY SHEAR* / *WEIGHT*		1*st STORY DRIFT* / *HEIGHT* (%)	
	L	T	L	T		L	T	L	T	L	T
MOTION #1	2.27	3.60	2.27	3.60	1.00	0.068	0.089	0.095	0.101	0.0513	0.0557
MOTION #2	2.86	5.88	2.86	5.88	1.00	0.077	0.112	0.054	0.065	0.0292	0.0357
MOTION #3	5.18	4.81	5.18	4.81	1.00	0.104	0.103	0.059	0.077	0.0318	0.0421
MOTION #4	4.12	6.92	4.12	6.92	1.00	0.089	0.113	0.088	0.111	0.0479	0.0606
MOTION #5	3.85	8.06	3.85	8.07	1.00	0.087	0.127	0.064	0.079	0.0379	0.0430
MOTION #6	9.49	11.17	9.49	11.17	1.00	0.149	0.152	0.077	0.086	0.0419	0.0470

TABLE 8.4 Analysis results for 1 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.10, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Weight is total weight of structure including base. Height is 12ft.

EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	$\frac{\textit{BASE SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{1st STORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{1st STORY DRIFT}}{\textit{HEIGHT}}$ (%)	
	L	T	L	T		L	T	L	T	L	T
MOTION #1	1.14	1.97	1.14	1.97	1.00	0.103	0.120	0.191	0.232	0.1035	0.1277
MOTION #2	1.43	2.68	1.43	2.69	1.00	0.107	0.119	0.101	0.115	0.0546	0.0633
MOTION #3	1.92	2.35	1.92	2.36	1.00	0.120	0.119	0.090	0.114	0.0489	0.0628
MOTION #4	1.84	4.01	1.84	4.02	1.00	0.114	0.134	0.118	0.155	0.0642	0.0853
MOTION #5	2.72	5.39	2.73	5.40	1.00	0.116	0.152	0.121	0.136	0.0654	0.0745
MOTION #6	5.05	6.23	5.07	6.25	1.00	0.134	0.152	0.081	0.144	0.0438	0.0785

TABLE 8.5 Analysis results for 8 - story isolated structure with R=39.132 in (T_b = 2 sec), f_{max} = 0.10, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Weight is total weight of structure including base. Height is 12ft.

EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	$\frac{\textit{BASE SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{1st STORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{1st STORY DRIFT}}{\textit{HEIGHT}}$ (%)	
	L	T	L	T		L	T	L	T	L	T
MOTION #1	1.22	2.51	1.44	2.64	1.05	0.116	0.158	0.111	0.152	0.2630	0.3606
MOTION #2	1.41	2.13	1.48	2.31	1.08	0.121	0.134	0.113	0.125	0.2652	0.2933
MOTION #3	2.06	2.61	2.21	2.76	1.06	0.133	0.139	0.125	0.132	0.2896	0.3180
MOTION #4	2.12	2.85	2.23	2.86	1.00	0.142	0.160	0.129	0.174	0.3048	0.4256
MOTION #5	2.53	5.52	2.60	5.58	1.01	0.126	0.229	0.126	0.223	0.3015	0.5292
MOTION #6	4.14	4.71	4.17	4.74	1.01	0.170	0.200	0.152	0.195	0.3537	0.4701

TABLE 8.6 Analysis results for 8 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.05, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Weight is total weight of structure including base. Height is 12ft.

EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR / WEIGHT*		1*st STORY SHEAR / WEIGHT*		1*st STORY DRIFT / HEIGHT* (%)	
	L	T	L	T		L	T	L	T	L	T
MOTION #1	2.75	3.66	2.76	3.70	1.01	0.079	0.089	0.069	0.085	0.1615	0.2020
MOTION #2	3.25	6.84	3.27	6.92	1.01	0.073	0.118	0.064	0.111	0.1494	0.2583
MOTION #3	5.39	4.75	5.50	4.84	1.02	0.109	0.094	0.094	0.088	0.2231	0.2121
MOTION #4	4.87	6.88	4.89	6.89	1.00	0.101	0.110	0.098	0.124	0.2307	0.2986
MOTION #5	4.16	6.56	4.17	6.68	1.02	0.092	0.109	0.082	0.108	0.1881	0.2567
MOTION #6	9.75	11.17	9.85	11.22	1.00	0.145	0.152	0.134	0.142	0.3100	0.3385

TABLE 8.7 Analysis results for 8 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.10, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Weight is total weight of structure including base. Height is 12ft.

EXCITATION	BASE CENTER DISPL (INCH)		CORNER BEARING DISPL (INCH)		CORNER TO CENTER DISPL RATIO	*BASE SHEAR* / *WEIGHT*		*1st STORY SHEAR* / *WEIGHT*		*1st STORY DRIFT* / *HEIGHT* (%)	
	L	T	L	T		L	T	L	T	L	T
MOTION #1	1.43	3.68	1.50	4.01	1.09	0.106	0.139	0.112	0.128	0.2618	0.3045
MOTION #2	1.53	2.62	1.66	2.76	1.05	0.112	0.119	0.102	0.123	0.2389	0.2935
MOTION #3	2.53	3.11	2.61	3.13	1.01	0.112	0.126	0.109	0.125	0.2540	0.3015
MOTION #4	2.63	3.21	2.77	3.41	1.06	0.117	0.128	0.120	0.143	0.2874	0.3538
MOTION #5	2.69	5.64	2.89	5.66	1.00	0.104	0.151	0.129	0.152	0.3046	0.3651
MOTION #6	5.86	6.32	5.90	6..34	1.00	0.138	0.156	0.133	0.153	0.3075	0.3762

TABLE 8.8 Summary of analysis results of maximum base displacement at geometric center of 1 - story isolated structure excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC and comparison of these displacements with the design displacements according to SEAOC static analysis procedure.

Soil Type	Sliding Isolation System Pyroperties.								
	R=39.132 in (T_b = 2 sec.) fmax=0.10			R=88.048 in (T_b = 3 sec.) fmax=0.05			R=88.048 in (T_b = 3 sec.) fmax=0.10		
	Analysis (inch)	SEAOC (inch)	Ratio *	Analysis (inch)	SEAOC (inch)	Ratio *	Analysis (inch)	SEAOC (inch)	Ratio *
S1	2.16	2.81	1.30	5.88	5.28	0.90	2.68	3.11	1.16
S2	5.69	5.72	1.01	11.17	10.19	0.91	6.23	6.32	1.01

* SEAOC/Analysis

TABLE 8.9 Summary of analysis results of maximum base displacement at geometric center of 8 - story isolated structure excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC and comparison of these displacements with the design displacements according to SEAOC static analysis procedure.

Soil Type	Sliding Isolation System Properties.								
	R=39.132 in (T_b = 2 sec.) fmax=0.10			R=88.048 in (T_b = 3 sec.) fmax=0.05			R=88.048 in (T_b = 3 sec.) fmax=0.10		
	Analysis (inch)	SEAOC (inch)	Ratio *	Analysis (inch)	SEAOC (inch)	Ratio *	Analysis (inch)	SEAOC (inch)	Ratio *
S1	2.61	2.81	1.08	6.84	5.28	0.77	3.68	3.11	0.85
S2	5.52	5.72	1.04	11.17	10.19	0.91	6.32	6.32	1.00

* SEAOC/Analysis

SECTION 9

CONCLUSIONS AND DISCUSSION

In this study, a comparison has been made between SEAOC design requirements and sliding isolated structure results obtained either by tests or by dynamic nonlinear time history analysis. In the dynamic analysis, six different combinations of structural systems and properties of isolation system were considered. The structural systems consisted of either a 1 - story stiff structure or an 8 - story flexible structure. The isolation system consisted of 45 Friction Pendulum System (FPS) isolators with stiffness and frictional properties covering a wide range of values. The isolators were modeled as elements having linear stiffness and friction with circular interaction. In this way, the force-displacement relation of each isolator was identical in all directions.

Each isolated structure was analyzed by three different procedures. In the first, a small set of artificial motions was used. These motions were comparable with design spectra for Seismic Zone 4. In the second, another small set of actual but scaled records was used. These records were also compatible with design spectra for Seismic Zone 4. The scaling of these records followed the procedure required by the SEAOC for time history analysis. In the third, a large set of actual earthquake records was used. The records were scaled so that the peak ground velocity (PGV) of each record had a value compatible with spectra for Seismic Zone 4. In this case, the variation in the response due to the variability of ground motion was evaluated by calculating mean and standard deviation values.

This study concentrated on the evaluation of the SEAOC static analysis formula that prescribes peak displacements of the isolation system. However, additional results like base shear force, story shear forces and interstory drifts are presented for all analyzed structures. This collection of nonlinear response data could be further used to evaluate design requirements for sliding isolated structures.

The conclusions of this study are:

(1)Friction pendulum bearings can be accurately modeled with bilinear (non-velocity dependent) hysteretic elements. In this respect, standard computer programs like DRAIN-2D may be used provided that care is exercised in selecting the proper "yield displacement" and time step for integration.

(2)The SEAOC formula for the design displacement (equation 2.1) overpredicts uni-directional test displacements. However, the amount of overprediction is difficult to quantify because of the difficulty in establishing ZNS values which are representative of a single earthquake motion history.

(3)The SEAOC formula (equation 2.1) overpredicts uni-directional artificial time history displacements by an average of about 50%. For the calculation of the time history displacements, three artificial (spectrum compatible) earthquake motions were used for each set of analyses. Furthermore, the SEAOC formula overpredicts bi-directional artificial time history displacements by an average of 25%. The bi-directional excitation consisted of one artificial, spectrum compatible earthquake motion applied in one building direction and 83% of the same motion applied in the other direction. In this way, the square root of the sum of squares of the spectra of the two artificial components was compatible with 1.3 times the Design-Basis spectra.

(4)The SEAOC formula (equation 2.1) predicts accurately the mean peak displacement response of several bi-directional real earthquakes scaled to have a common PGV and whose average spectrum equals the SEAOC design spectrum. Furthermore, the scaled earthquakes have the average of their SRSS combined spectra (square root of sum of squares of the L and T spectra) above the 1.3 times the SEAOC design spectrum (Design-Basis spectrum).

(5)The SEAOC formula (equation 2.1) predicts accurately the peak displacement response as calculated by the time history analysis method specified by SEAOC for dynamic analysis.

(6)The additional displacement due to torsion is significantly lower in sliding isolation systems than in other isolation systems. In particular, a 5% mass eccentricity in a stiff 1 - story structure was found to generate, in all analyzed cases, an insignificant additional displacement in sliding isolation systems. The maximum calculated ratio of corner bearing displacement to geometric center displacement was only 1.02, whereas the SEAOC static procedure prescribes a value of 1.24 for the analyzed plan configuration. In the case of a flexible 8 - story structure, the additional displacement due to torsion is considerably larger than that in the stiff 1 - story structure. In general, torsion in sliding isolation systems is primarily affected by the combination of mass eccentricity and superstructure flexibility and not by the mass eccentricity alone. In this respect, the minimum factor of 1.1 specified in SEAOC for the amplification of the design displacement (D) to account for torsion should be modified so that it reflects the effect of the superstructure flexibility.

The main conclusion of this study is that the SEAOC static analysis procedure predicts displacements of the isolation system which compare well with displacements calculated in time history dynamic analysis. In this analysis, the earthquake motions consisted of two orthogonal components whose spectra, when combined by the SRSS rule, matched or were above the 1.3 times the SEAOC design spectra (Design-Basis spectra).

In the cases in which the earthquake motions matched the 1.3 times the Design-Basis spectra (artificial records), the SEAOC formula overpredicted the time history displacements by about 25%.

In the cases in which the earthquake motions had combined spectra above the 1.3 times the Design-Basis spectra (PGV scaled, Figures 7-2 and 7-3), the SEAOC formula predicted well the mean peak displacement of these earthquake motions. When a square interaction model was used for the isolation bearings (as done in the study of Kircher and Lashkari, 1989), the SEAOC formula overpredicted the mean peak displacement by less than 20%. In this respect, the degree of conservatism in the SEAOC static analysis procedure appears to be about the same for the studied sliding isolation systems and the bilinear isolation systems studied by Kircher and Lashkari, 1989.

SECTION 10

REFERENCES

[1] Applied Technology Council, (1978). *Tentative Provisions for the Development of Seismic Regulations for Buildings,* ATC 3-06, NBS 510, NSF 78-8, Washington, D. C., June.

[2] California Institute of Technology (1975). *Analyses of Strong Motion Earthquake Accelerograms.* Report No. EERL 75-80, Vol III, Part T, Earthquake Engineering Laboratory, Pasadena, CA.

[3] Chalhoub, M.S. and Kelly, J.M. (1990). *Comparison of SEAONC Base Isolation Tentative Code to Shake Table Results*. Journal of Structural Engineering, ASCE, Vol. 116, No.4, pp.925-938.

[4] Constantinou, M.C., Mokha, A. and Reinhorn, A.M. (1990a). *Experimental and Analytical Study of a Combined Sliding Disc Bearing and Helical Steel Spring Isolation System*. Report No. NCEER-90-0019, National Center for Earthquake Engineering Research, Buffalo, N.Y.

[5] Constantinou, M.C., Mokha, A. and Reinhorn, A.M. (1990b). *Teflon Bearings in Base Isolation.* Part 2: Modeling. Journal of Structural Engineering, ASCE, Vol. 116, No.2, pp.455-474.

[6] Constantinou, M.C., Mokha,A. and Reinhorn, A.M. (1991).*Study of a Sliding Bearing and Helical Steel Spring Isolation System* . Journal of Structural Engineering, ASCE, Vol. 117, No.4, pp.1257-1275.

[7] Gasparini, D.A. and Vanmarke, E.H. (1976). *Simulated Earthquake Motions Compatible with Prescribed Response Spectra.* Report No. 2, Department of Civil Engineering,Massachusetts Institute of Technology, Boston, Massachussetts.

[8] Gear, C.W. (1971). *The Automatic Integration of Ordinary Differential Equations.* Numerical Mathematics, Communications of ACM, Vol. 14(3), pp.176-190.

[9] Griffith, M.C., Aiken, I.D. and Kelly, J.M. (1988). *Experimental Evaluation of Seismic Isolation of a 9 - Story Braced Steel Frame Subject to Uplift*. Report No. UBC/EERC-88/05. Earthquake Engineering Research Center, University of California, Berkeley, CA.

[10] Kircher, C.A. And Lashkari, B. (1989). *Statistical Evaluation of Nonlinear Response of Seismic Isolation Systems*. Report No. JBA 109-070, Jack R. Benjamin and Associates, Inc. Mountain View, CA.

[11] Mokha, A., Constantinou, M.C. and Reinhorn, A.M. (1988). *Teflon Bearings in Aseismic, Base Isolation. Experimental Studies and Mathematical Modeling*. Report No. NCEER-88-0038, National Center for Earthquake Engineering Research, State University of New York, Buffalo, N.Y.

[12] Mokha, A., Constantinou, M.C. and Reinhorn, A.M. (1990a). *Teflon Bearings in Base Isolation. Part 1: Testing*. Journal of Structural Engineering, ASCE, Vol. 116, No.2, pp.438-454.

[13] Mokha, A., Constantinou, M.C. and Reinhorn, A.M. (1990b). *Experimental and Analytical Study of a Sliding Isolation System with a Spherical Surface.*. Report No. NCEER-900020, National Center for Earthquake Engineering Research, Buffalo, N.Y.

[14] Mokha, A., Constantinou M.C., Reinhorn A.M. and Zayas V. (1991). *Experimental Study of Friction Pendulum Isolation System*. Journal of Structural Engineering, ASCE, Vol. 117, No.4, pp.1201-1217.

[15] Nagarajaiah, S., Reinhorn, A.M. and Constantinou, M.C. (1989). *Nonlinear Dynamic Analysis of Three - Dimensional Base Isolated Structures - Program 3D-BASIS*. Report No. NCEER-89-0019, National Center for Earthquake Engineering Research, State University of New York, Buffalo, N.Y.

[16] Powell, G.H. (1973). *DRAIN 2D User Guide*. Report No. EERC 73-22. Earthquake Engineering Research Center, University of California, Berkeley, CA.

[17] Structural Engineers Association of California (1990a). *Tentative General Requirements for the Design and Construction of Seismic Isolated Sructures*. Appendix IL of Recommended Lateral Force Requirements and Commentary. San Francisco, CA.

[18] Structural Engineers Association of California (1990b). *Recommended Lateral Force Requirements and Commentary (Blue Book)*. Seismology Committee.San Francisco, CA.

[19] Structural Engineers Association of Northern California (1986).*Tentative Seismic Isolation Design Requirements*, San Francisco, CA.

[20] Seed, H.B., Ugas, C. And Lysmer, J. (1974). *Site - Dependent Spectra for Earthquake Resistant Design*, Report No. EERC 74-12, Earthquake Engineering Research Center, University of California, Berkeley, CA.

[21] Uniform Building Code (1991). *International Congress of Building Officials*, Whittier, CA.

[22] Zayas, V., Low, S.S. and Mahin, S.A. (1987). *The FPS Earthquake Resisting System,* Experimental Report. Report No. UBC/EERC-87/01, Earthquake Engineering Research Center, University of California, Berkeley, CA.

APPENDIX A

RESULTS OF MAXIMUM STORY SHEAR AND INTERSTORY DRIFT FOR 8 - STORY ISOLATED STRUCTURE. EXCITATION REPRESENTED BY ARTIFICIAL RECORDS COMPATIBLE WITH DESIGN SPECTRA. EXCITATION ONLY IN THE TRANSVERSE (T) DIRECTION.

TABLE A.1 Analysis results of maximum story shear for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by artificial records compatible with Design Spectra. Weight is total weight of structure including base.

SOIL TYPE	EXCITATION	$\frac{\textit{2nd STORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{3rd STORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{4th STORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{5th STORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{6th STORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{7th STORY SHEAR}}{\textit{WEIGHT}}$		$\frac{\textit{8th STORY SHEAR}}{\textit{WEIGHT}}$	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
S1	ARTIFICIAL #1	0.000	0.119	0.000	0.111	0.000	0.103	0.000	0.097	0.000	0.079	0.000	0.091	0.000	0.064
S1	ARTIFICIAL #2	0.000	0.141	0.000	0.124	0.000	0.117	0.000	0.120	0.000	0.106	0.000	0.091	0.000	0.067
S1	ARTIFICIAL #3	0.000	0.127	0.000	0.112	0.000	0.092	0.000	0.084	0.000	0.090	0.000	0.094	0.000	0.070
S2	ARTIFICIAL #4	0.000	0.134	0.000	0.127	0.000	0.119	0.000	0.114	0.000	0.091	0.000	0.083	0.000	0.062
S2	ARTIFICIAL #5	0.000	0.139	0.000	0.145	0.000	0.141	0.000	0.136	0.000	0.117	0.000	0.097	0.000	0.065
S2	ARTIFICIAL #6	0.000	0.137	0.000	0.133	0.000	0.127	0.000	0.110	0.000	0.106	0.000	0.098	0.000	0.063
S3	ARTIFICIAL #7	0.000	0.225	0.000	0.202	0.000	0.186	0.000	0.146	0.000	0.141	0.000	0.121	0.000	0.082
S3	ARTIFICIAL #8	0.000	0.180	0.000	0.166	0.000	0.153	0.000	0.125	0.000	0.120	0.000	0.112	0.000	0.076
S3	ARTIFICIAL #9	0.000	0.198	0.000	0.180	0.000	0.159	0.000	0.143	0.000	0.137	0.000	0.122	0.000	0.080

TABLE A.2 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by artificial records compatible with Design Spectra. Height is 12ft.

SOIL TYPE	EXCITATION	$\frac{\text{2nd STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{3rd STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{4th STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{5th STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{6th STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{7th STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{8th STORY DRIFT}}{\text{HEIGHT}}(\%)$	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
S1	ARTIFICIAL #1	0.0000	0.2860	0.0000	0.2669	0.0000	0.3301	0.0000	0.3095	0.0000	0.2554	0.0000	0.4323	0.0000	0.3068
S1	ARTIFICIAL #2	0.0000	0.3243	0.0000	0.2910	0.0000	0.3673	0.0000	0.3705	0.0000	0.3242	0.0000	0.4324	0.0000	0.3176
S1	ARTIFICIAL #3	0.0000	0.2977	0.0000	0.2686	0.0000	0.2932	0.0000	0.2726	0.0000	0.2870	0.0000	0.4517	0.0000	0.3300
S2	ARTIFICIAL #4	0.0000	0.3169	0.0000	0.2993	0.0000	0.3834	0.0000	0.3542	0.0000	0.2823	0.0000	0.3970	0.0000	0.2931
S2	ARTIFICIAL #5	0.0000	0.3167	0.0000	0.3378	0.0000	0.4473	0.0000	0.4307	0.0000	0.3722	0.0000	0.4615	0.0000	0.3125
S2	ARTIFICIAL #6	0.0000	0.3189	0.0000	0.3184	0.0000	0.4043	0.0000	0.3494	0.0000	0.3357	0.0000	0.4669	0.0000	0.3000
S3	ARTIFICIAL #7	0.0000	0.5377	0.0000	0.4890	0.0000	0.6040	0.0000	0.4599	0.0000	0.4384	0.0000	0.5834	0.0000	0.3913
S3	ARTIFICIAL #8	0.0000	0.4364	0.0000	0.3971	0.0000	0.5027	0.0000	0.3973	0.0000	0.3782	0.0000	0.5299	0.0000	0.3615
S3	ARTIFICIAL #9	0.0000	0.4800	0.0000	0.4269	0.0000	0.5032	0.0000	0.4456	0.0000	0.4286	0.0000	0.5789	0.0000	0.3807

TABLE A.3 Analysis results of maximum story shear for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by artificial records compatible with Design Spectra. Weight is total weight of structure including base.

SOIL TYPE	EXCITATION	2nd STORY SHEAR / WEIGHT		3rd STORY SHEAR / WEIGHT		4th STORY SHEAR / WEIGHT		5th STORY SHEAR / WEIGHT		6th STORY SHEAR / WEIGHT		7th STORY SHEAR / WEIGHT		8th STORY SHEAR / WEIGHT	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
S1	ARTIFICIAL #1	0.000	0.086	0.000	0.088	0.000	0.078	0.000	0.069	0.000	0.074	0.000	0.069	0.000	0.048
S1	ARTIFICIAL #2	0.000	0.081	0.000	0.072	0.000	0.062	0.000	0.061	0.000	0.053	0.000	0.053	0.000	0.041
S1	ARTIFICIAL #3	0.000	0.086	0.000	0.074	0.000	0.072	0.000	0.068	0.000	0.068	0.000	0.059	0.000	0.051
S2	ARTIFICIAL #4	0.000	0.122	0.000	0.117	0.000	0.110	0.000	0.096	0.000	0.078	0.000	0.063	0.000	0.041
S2	ARTIFICIAL #5	0.000	0.113	0.000	0.116	0.000	0.119	0.000	0.104	0.000	0.098	0.000	0.073	0.000	0.055
S2	ARTIFICIAL #6	0.000	0.114	0.000	0.112	0.000	0.107	0.000	0.095	0.000	0.080	0.000	0.060	0.000	0.047
S3	ARTIFICIAL #7	0.000	0.123	0.000	0.120	0.000	0.110	0.000	0.096	0.000	0.093	0.000	0.085	0.000	0.052
S3	ARTIFICIAL #8	0.000	0.138	0.000	0.118	0.000	0.103	0.000	0.096	0.000	0.082	0.000	0.070	0.000	0.054
S3	ARTIFICIAL #9	0.000	0.179	0.000	0.163	0.000	0.146	0.000	0.122	0.000	0.097	0.000	0.085	0.000	0.055

TABLE A.4 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by artificial records compatible with Design Spectra. Height is 12ft.

SOIL TYPE	EXCITATION	$\frac{\textit{2nd STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{3rd STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{4th STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{5th STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{6th STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{7th STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{8th STORY DRIFT}}{\textit{HEIGHT}}$ (%)	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
S1	ARTIFICIAL #1	0.0000	0.2033	0.0000	0.2019	0.0000	0.2445	0.0000	0.2213	0.0000	0.2395	0.0000	0.3342	0.0000	0.02263
S1	ARTIFICIAL #2	0.0000	0.1896	0.0000	0.1696	0.0000	0.2029	0.0000	0.1940	0.0000	0.1646	0.0000	0.2478	0.0000	0.01995
S1	ARTIFICIAL #3	0.0000	0.2011	0.0000	0.1765	0.0000	0.2263	0.0000	0.2077	0.0000	0.2149	0.0000	0.2778	0.0000	0.02426
S2	ARTIFICIAL #4	0.0000	0.2884	0.0000	0.2779	0.0000	0.3450	0.0000	0.3036	0.0000	0.2494	0.0000	0.2994	0.0000	0.01937
S2	ARTIFICIAL #5	0.0000	0.2657	0.0000	0.2725	0.0000	0.3701	0.0000	0.3291	0.0000	0.3075	0.0000	0.3460	0.0000	0.02571
S2	ARTIFICIAL #6	0.0000	0.2714	0.0000	0.2636	0.0000	0.3350	0.0000	0.3007	0.0000	0.2547	0.0000	0.2865	0.0000	0.02247
S3	ARTIFICIAL #7	0.0000	0.2992	0.0000	0.2779	0.0000	0.3475	0.0000	0.3059	0.0000	0.2892	0.0000	0.4066	0.0000	0.02535
S3	ARTIFICIAL #8	0.0000	0.3189	0.0000	0.2757	0.0000	0.3287	0.0000	0.2991	0.0000	0.2565	0.0000	0.3353	0.0000	0.02562
S3	ARTIFICIAL #9	0.0000	0.4239	0.0000	0.3865	0.0000	0.4624	0.0000	0.3887	0.0000	0.3071	0.0000	0.4048	0.0000	0.02654

TABLE A.5 **Analysis results of maximum story shear for 8 - story isolated structure with R = 88.148 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by artificial records compatible with Design Spectra. Weight is total weight of structure including base.**

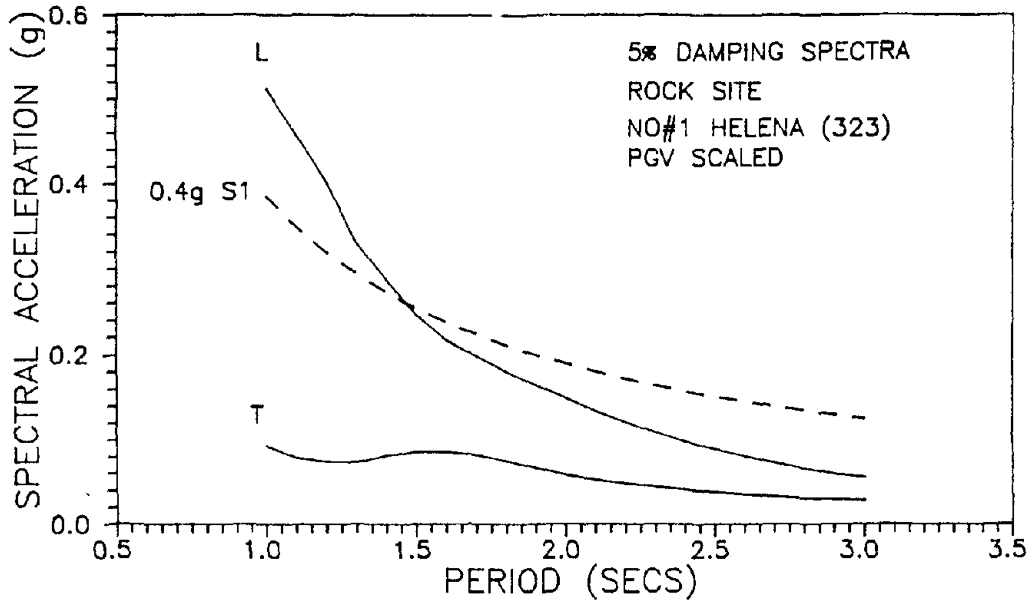
SOIL TYPE	EXCITATION	*2nd STORY SHEAR / WEIGHT*		*3rd STORY SHEAR / WEIGHT*		*4th STORY SHEAR / WEIGHT*		*5th STORY SHEAR / WEIGHT*		*6th STORY SHEAR / WEIGHT*		*7th STORY SHEAR / WEIGHT*		*8th STORY SHEAR / WEIGHT*	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
S1	ARTIFICIAL #1	0.000	0.109	0.000	0.100	0.000	0.101	0.000	0.101	0.000	0.090	0.000	0.099	0.000	0.067
S1	ARTIFICIAL #2	0.000	0.139	0.000	0.124	0.000	0.118	0.000	0.122	0.000	0.101	0.000	0.088	0.000	0.064
S1	ARTIFICIAL #3	0.000	0.127	0.000	0.111	0.000	0.090	0.000	0.098	0.000	0.085	0.000	0.092	0.000	0.069
S2	ARTIFICIAL #4	0.000	0.114	0.000	0.113	0.000	0.122	0.000	0.120	0.000	0.097	0.000	0.089	0.000	0.061
S2	ARTIFICIAL #5	0.000	0.153	0.000	0.161	0.000	0.153	0.000	0.141	0.000	0.122	0.000	0.101	0.000	0.069
S2	ARTIFICIAL #6	0.000	0.122	0.000	0.119	0.000	0.122	0.000	0.110	0.000	0.103	0.000	0.099	0.000	0.063
S3	ARTIFICIAL #7	0.000	0.160	0.000	0.166	0.000	0.156	0.000	0.138	0.000	0.127	0.000	0.112	0.000	0.077
S3	ARTIFICIAL #8	0.000	0.167	0.000	0.159	0.000	0.140	0.000	0.128	0.000	0.120	0.000	0.110	0.000	0.076
S3	ARTIFICIAL #9	0.000	0.177	0.000	0.159	0.000	0.140	0.000	0.132	0.000	0.128	0.000	0.116	0.000	0.081

TABLE A.6 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 88.148 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by artificial records compatible with Design Spectra. Height is 12ft.

SOIL TYPE	EXCITATION	$\frac{\text{2nd STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{3rd STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{4th STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{5th STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{6th STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{7th STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{8th STORY DRIFT}}{\text{HEIGHT}}$(%)	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
S1	ARTIFICIAL #1	0.0000	0.2546	0.0000	0.2376	0.0000	0.3140	0.0000	0.3117	0.0000	0.2814	0.0000	0.4697	0.0000	0.3217
S1	ARTIFICIAL #2	0.0000	0.3182	0.0000	0.2931	0.0000	0.3679	0.0000	0.3766	0.0000	0.3122	0.0000	0.4103	0.0000	0.3037
S1	ARTIFICIAL #3	0.0000	0.2952	0.0000	0.2681	0.0000	0.2880	0.0000	0.3014	0.0000	0.2653	0.0000	0.4431	0.0000	0.3279
S2	ARTIFICIAL #4	0.0000	0.2692	0.0000	0.2708	0.0000	0.3811	0.0000	0.3765	0.0000	0.3042	0.0000	0.4262	0.0000	0.2921
S2	ARTIFICIAL #5	0.0000	0.3499	0.0000	0.3746	0.0000	0.4831	0.0000	0.4490	0.0000	0.3860	0.0000	0.4779	0.0000	0.3261
S2	ARTIFICIAL #6	0.0000	0.2942	0.0000	0.2860	0.0000	0.3802	0.0000	0.3433	0.0000	0.3267	0.0000	0.4725	0.0000	0.2980
S3	ARTIFICIAL #7	0.0000	0.3662	0.0000	0.3880	0.0000	0.4985	0.0000	0.4386	0.0000	0.4013	0.0000	0.5322	0.0000	0.3664
S3	ARTIFICIAL #8	0.0000	0.3947	0.0000	0.3799	0.0000	0.4449	0.0000	0.3972	0.0000	0.3778	0.0000	0.5196	0.0000	0.3635
S3	ARTIFICIAL #9	0.0000	0.4089	0.0000	0.3758	0.0000	0.4482	0.0000	0.4117	0.0000	0.3981	0.0000	0.5523	0.0000	0.3855

APPENDIX B

RESPONSE SPECTRA FOR 5% DAMPING OF COMPONENTS OF PGV SCALED MOTIONS USED IN DYNAMIC ANALYSES AND COMPARISON TO DESIGN SPECTRA.



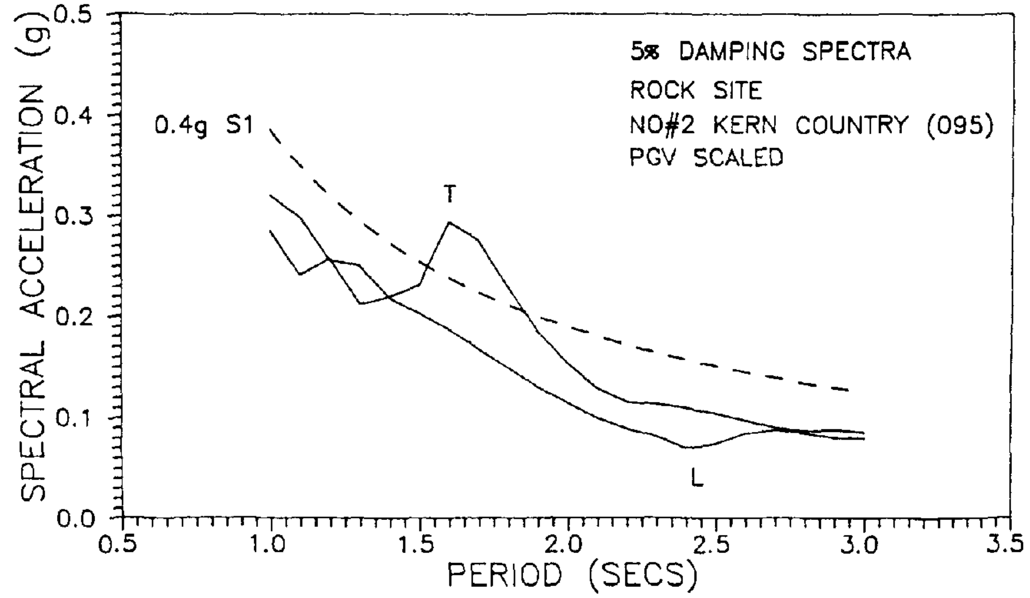
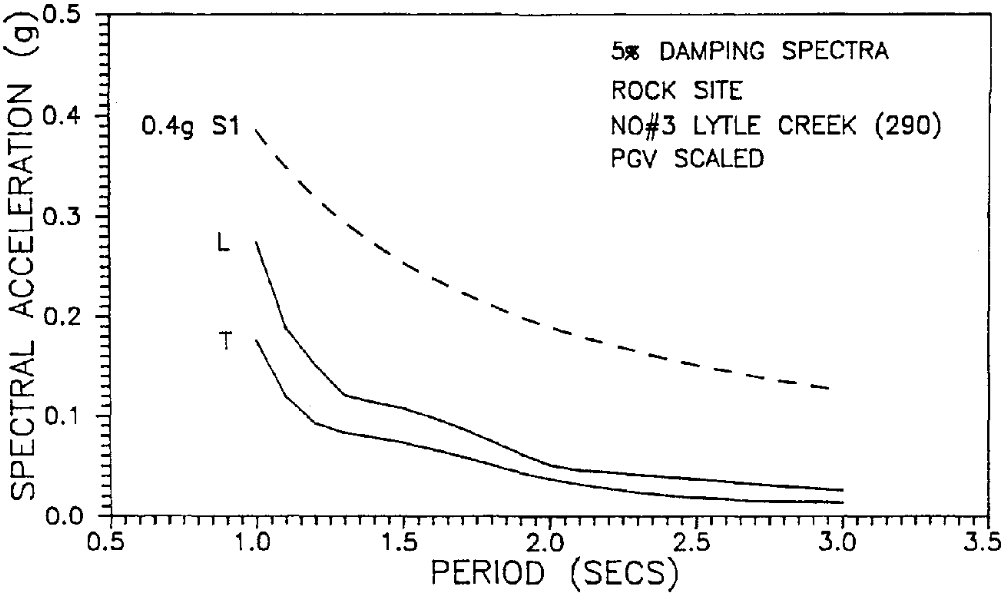


FIGURE B-1 **Response spectra for 5% damping of components of PGV scaled motions used in dynamic analyses and comparison to Design Spectra.**



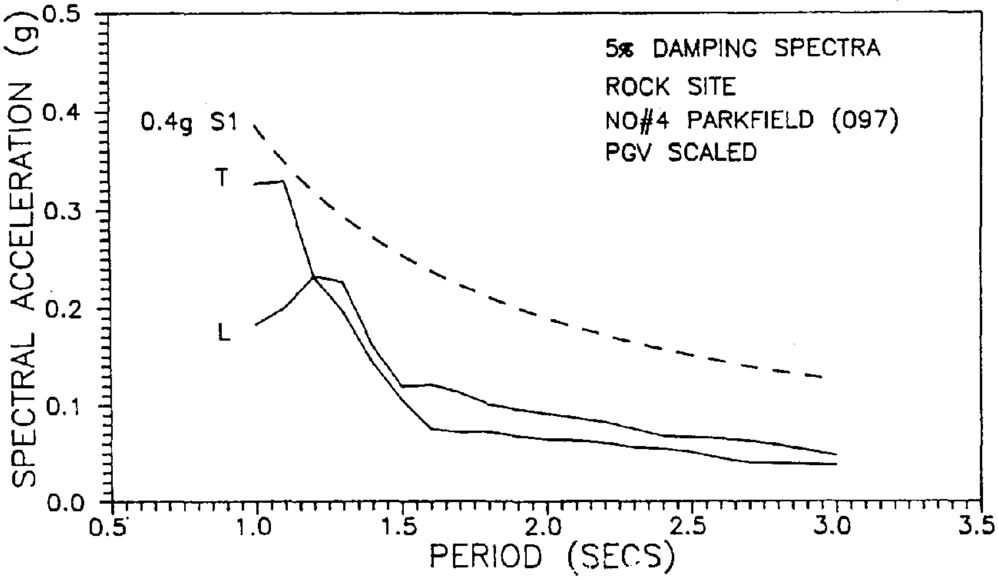
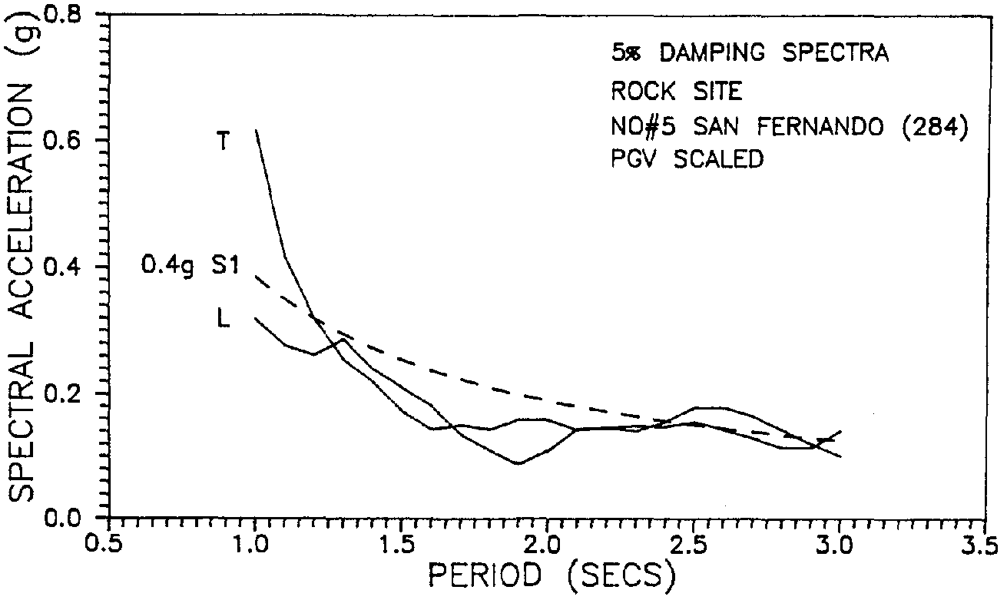


FIGURE B-1 Continued.



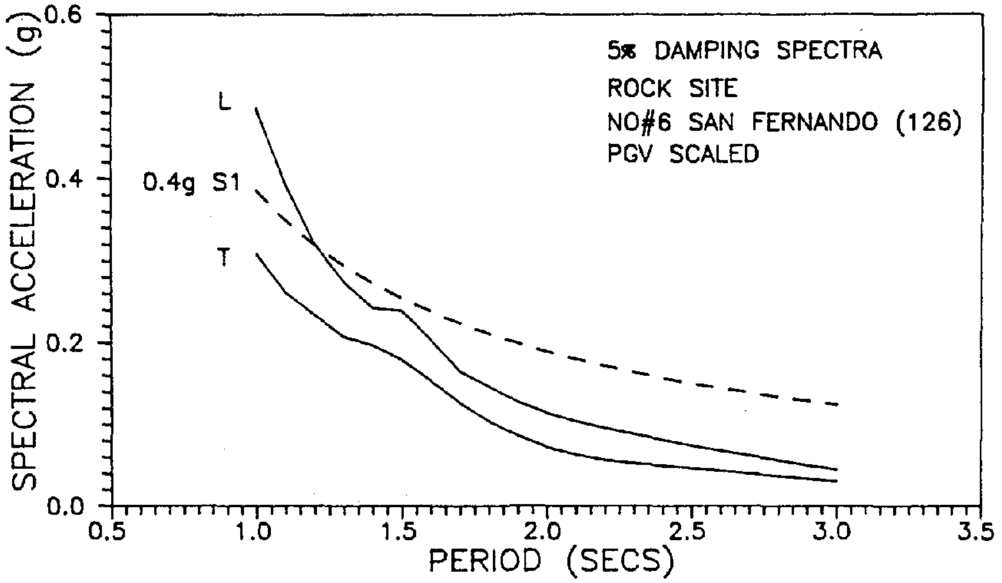
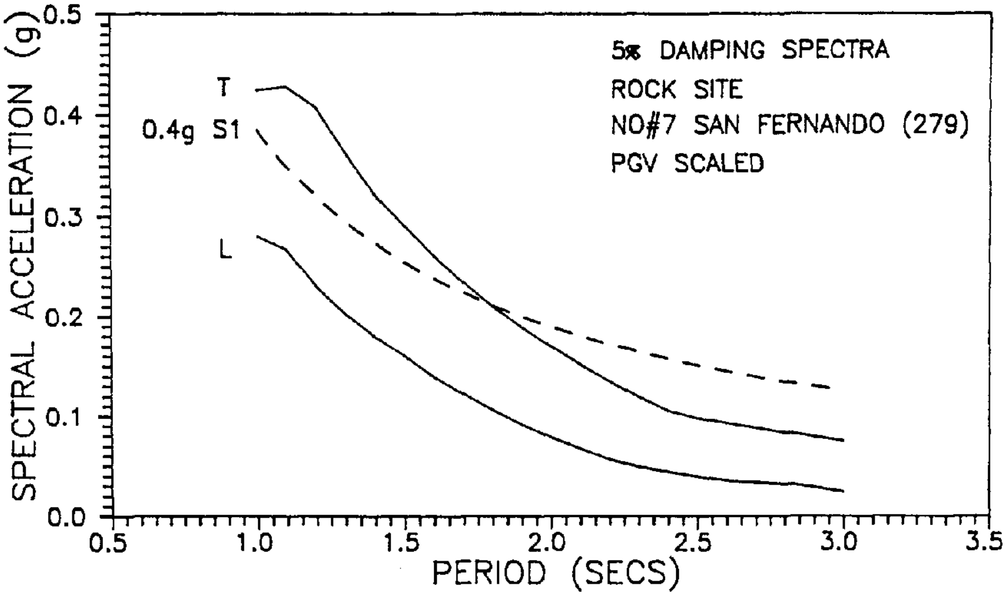


FIGURE B-1 Continued.



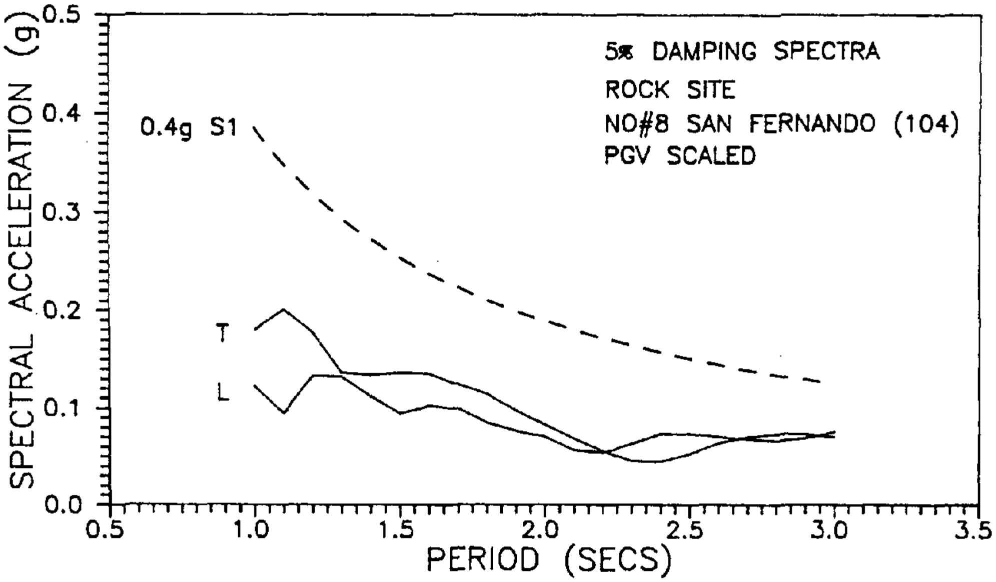
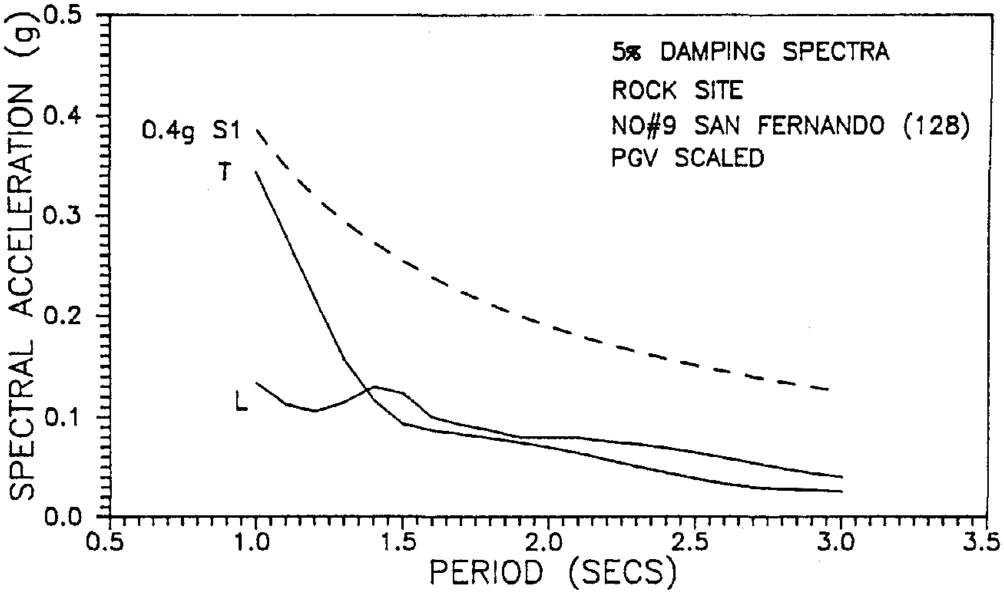


FIGURE B-1 Continued.



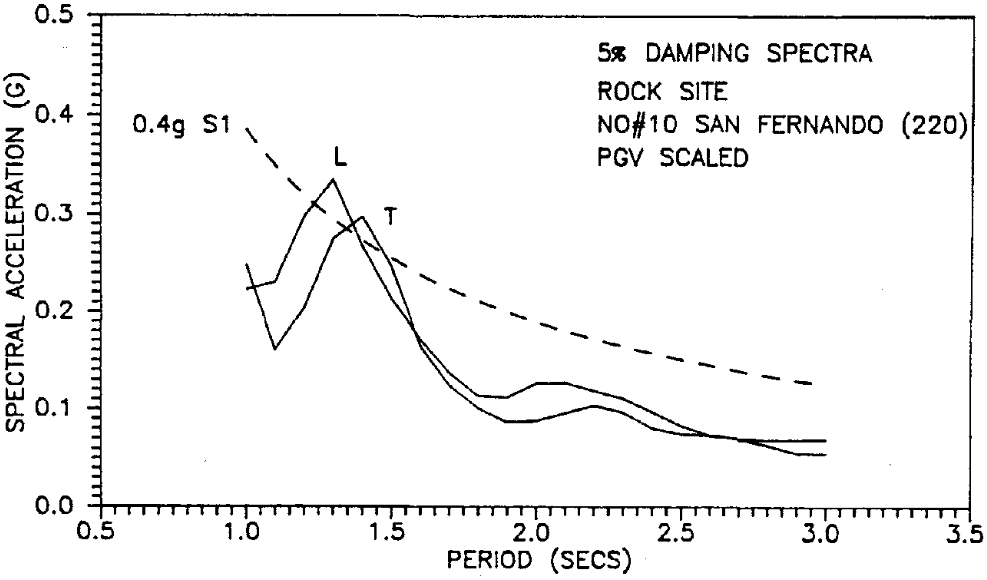
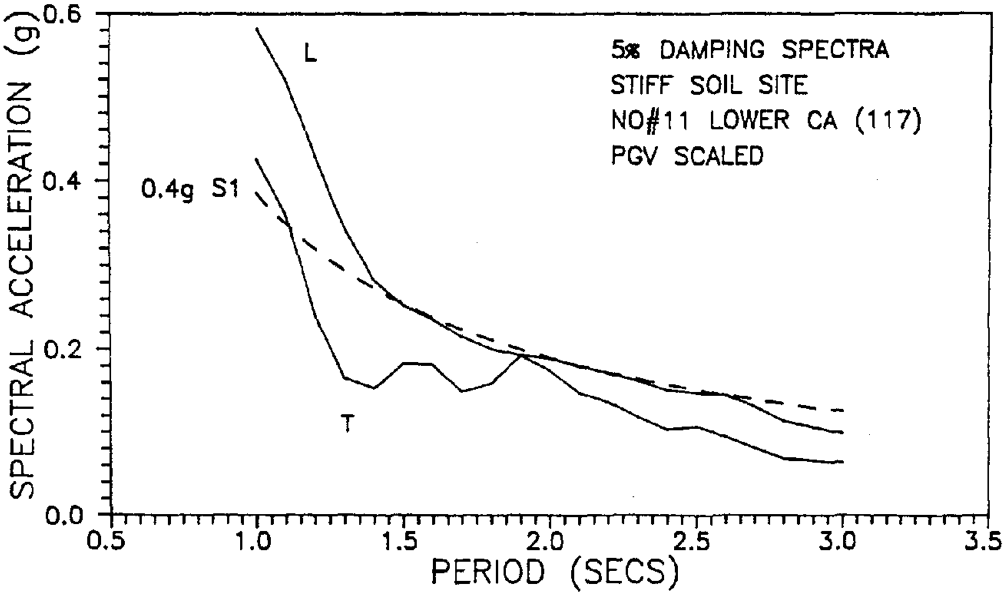


FIGURE B-1 Continued.



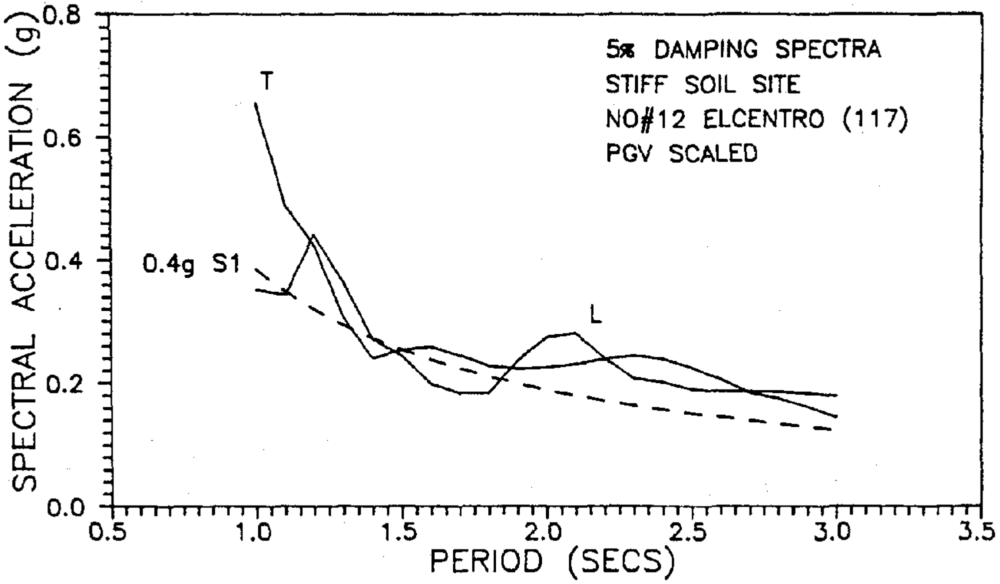
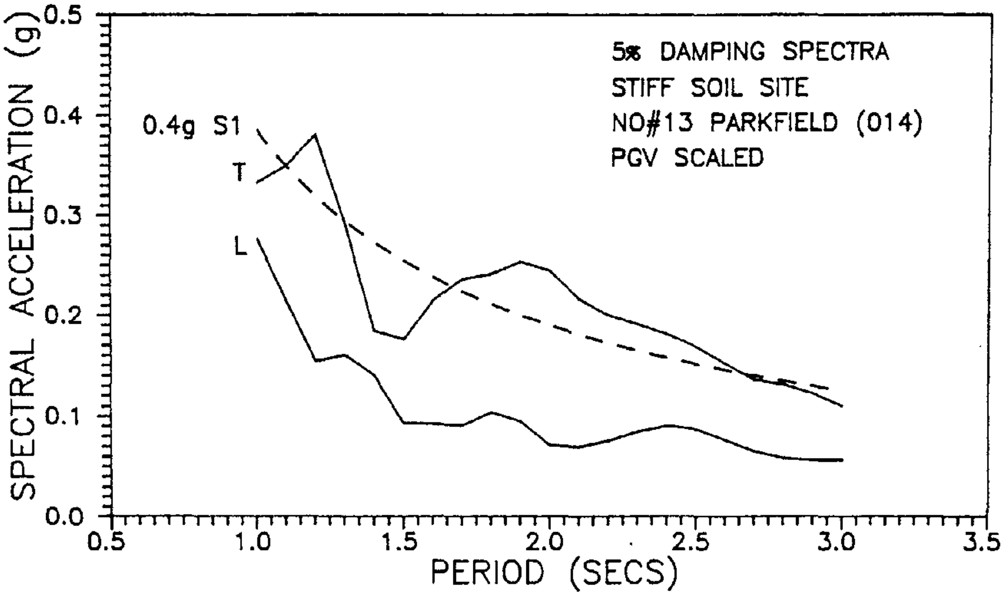


FIGURE B-1 Continued.



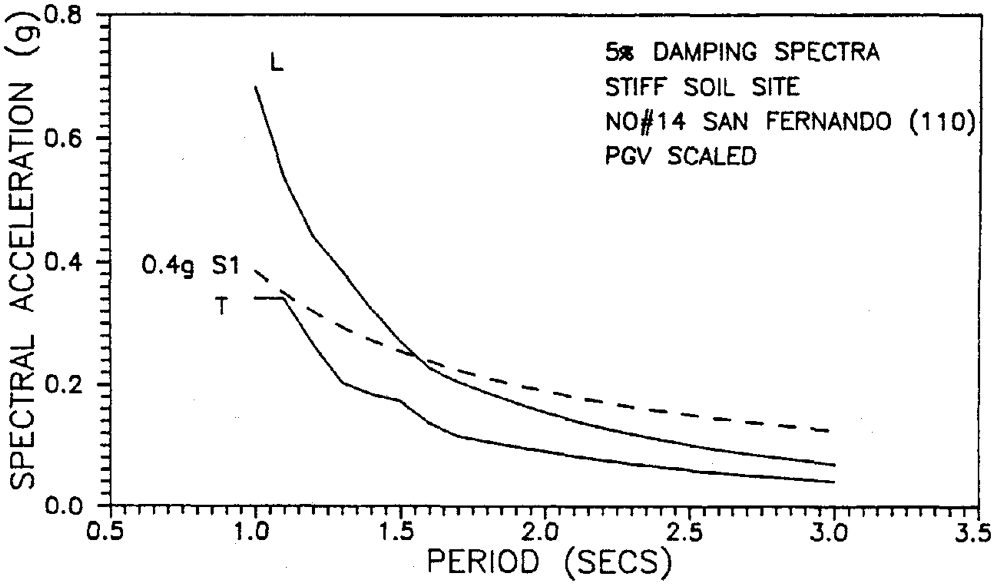
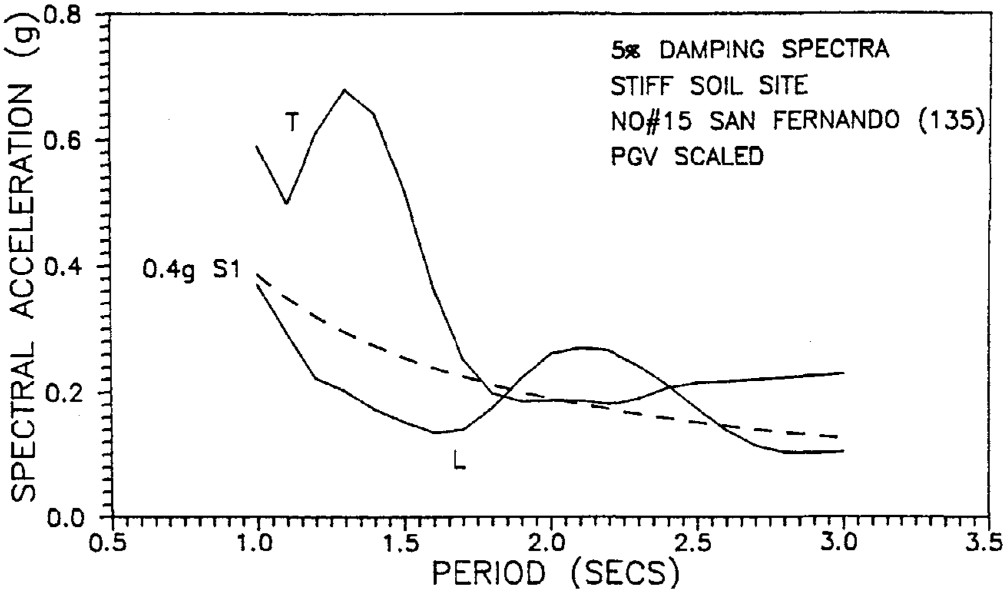


FIGURE B-1 Continued.



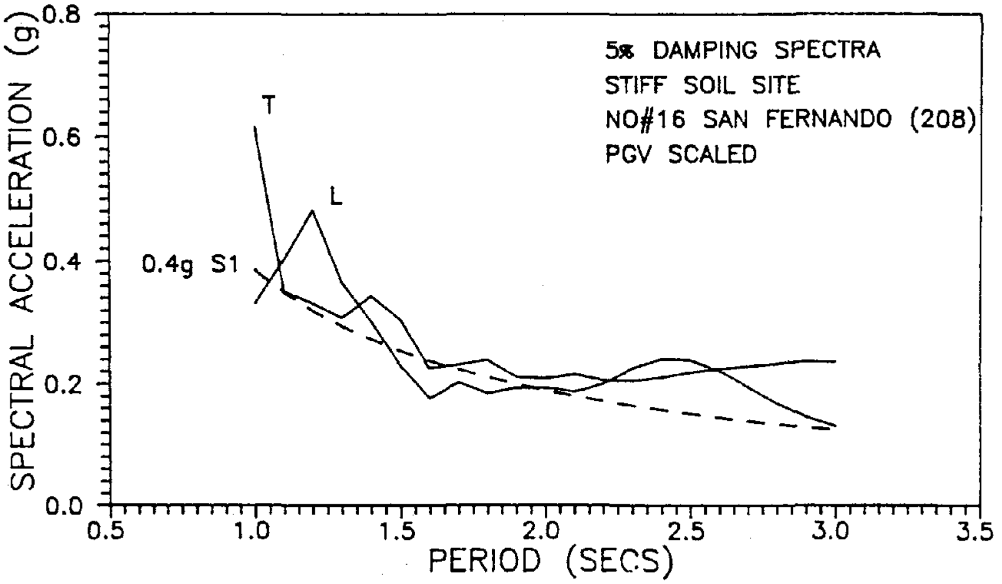
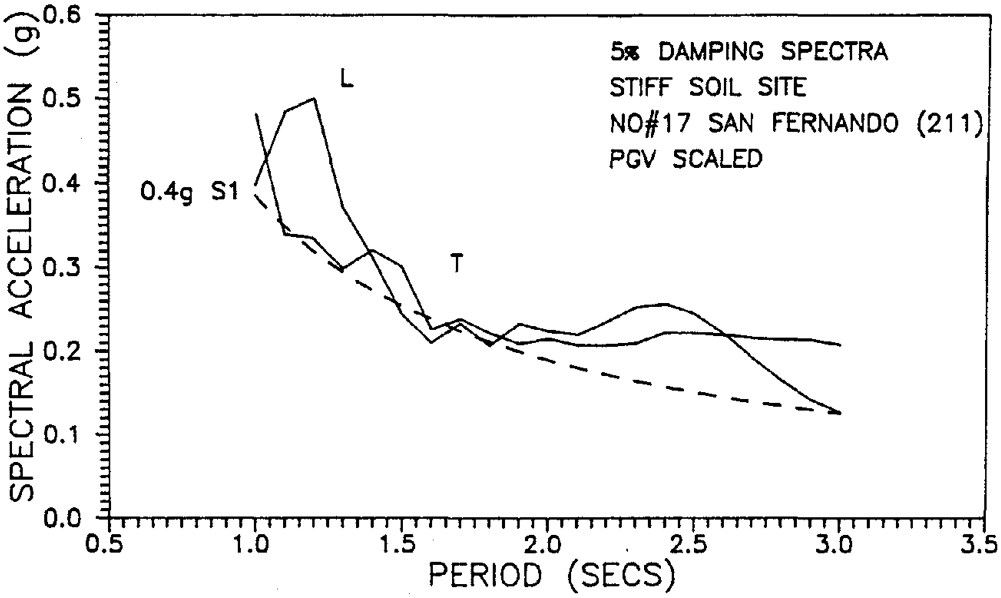


FIGURE B-1 Continued.



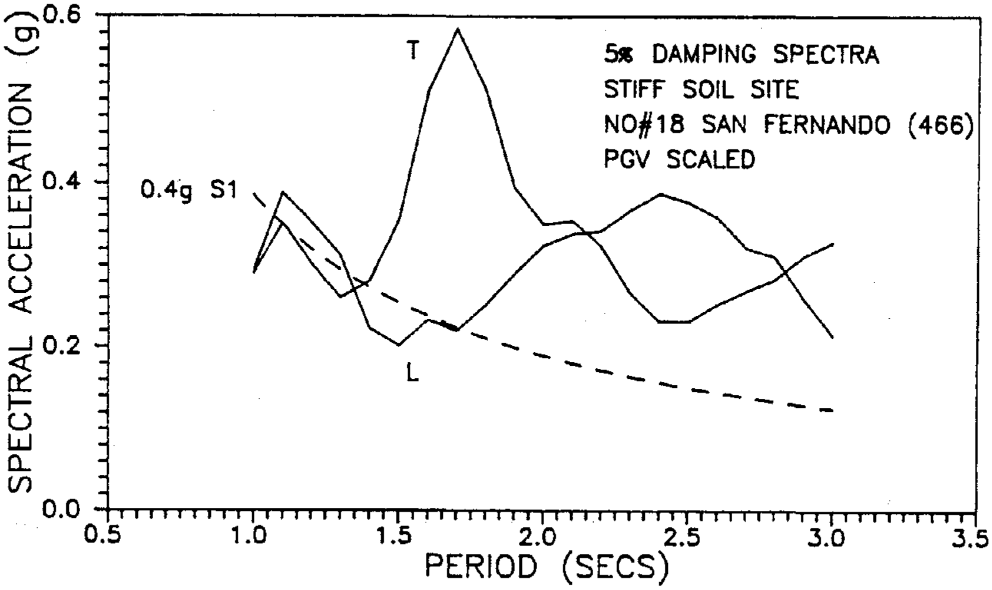
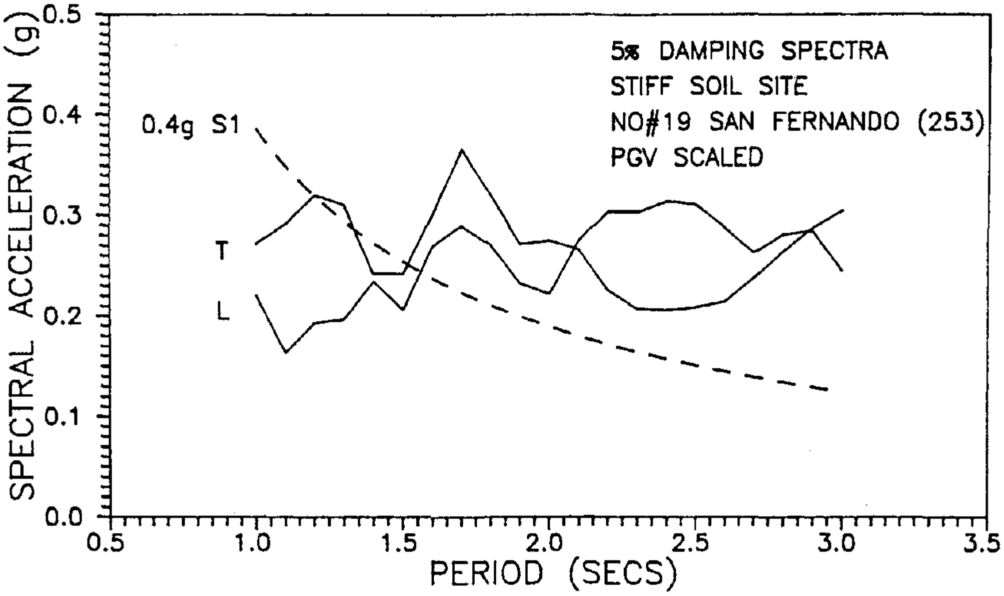


FIGURE B-1 Continued.



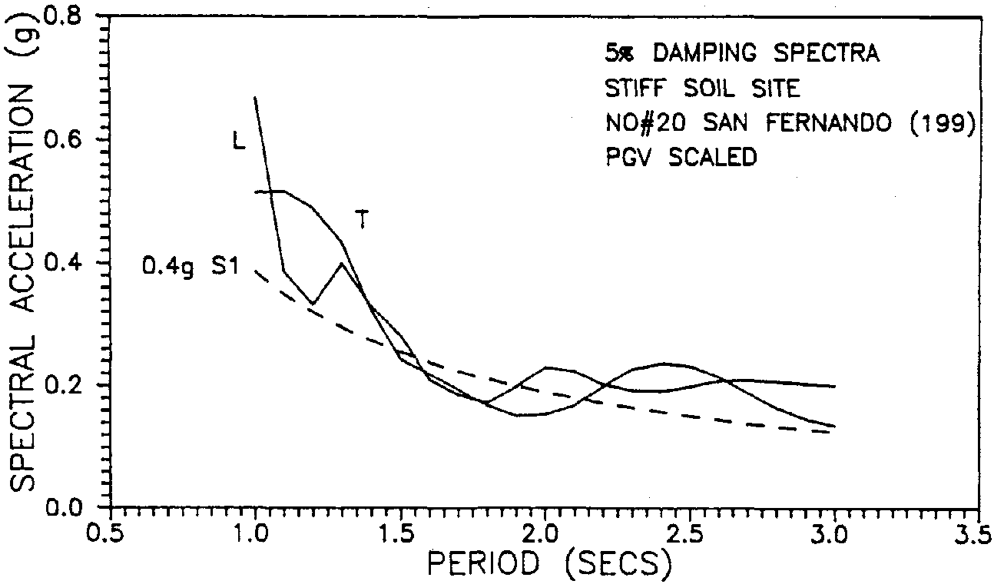
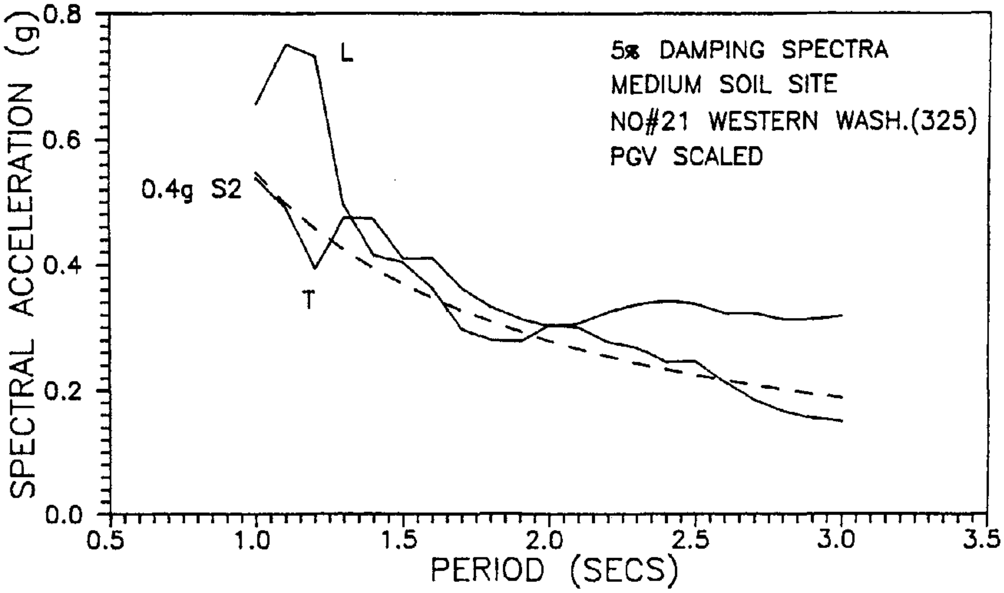


FIGURE B-1 Continued.



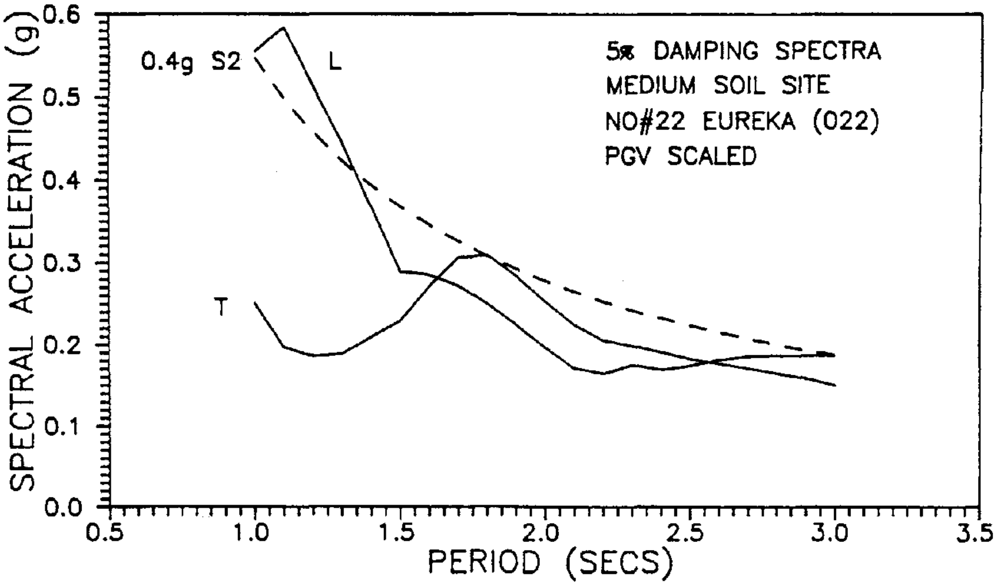
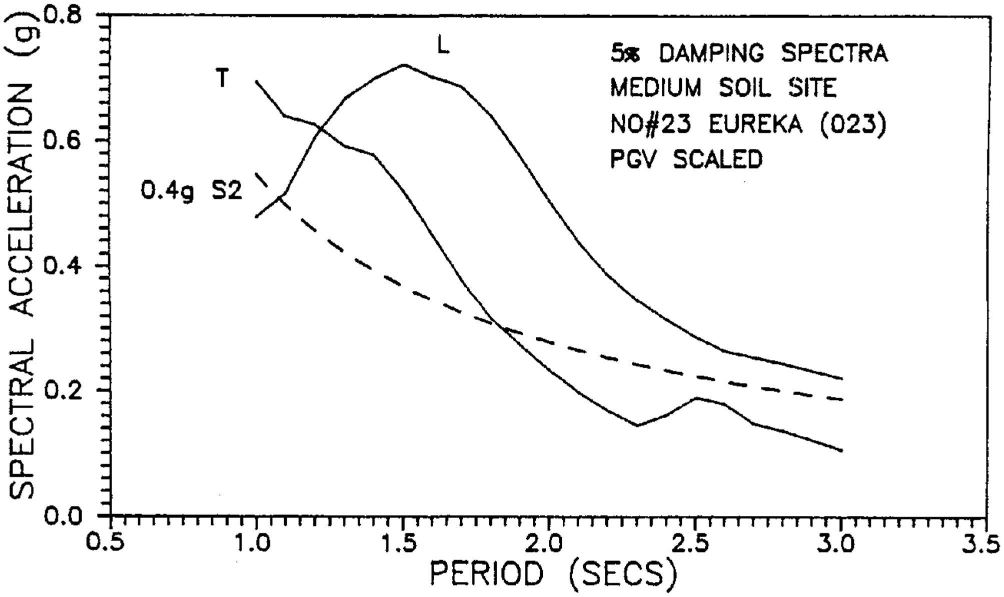


FIGURE B-1 Continued.



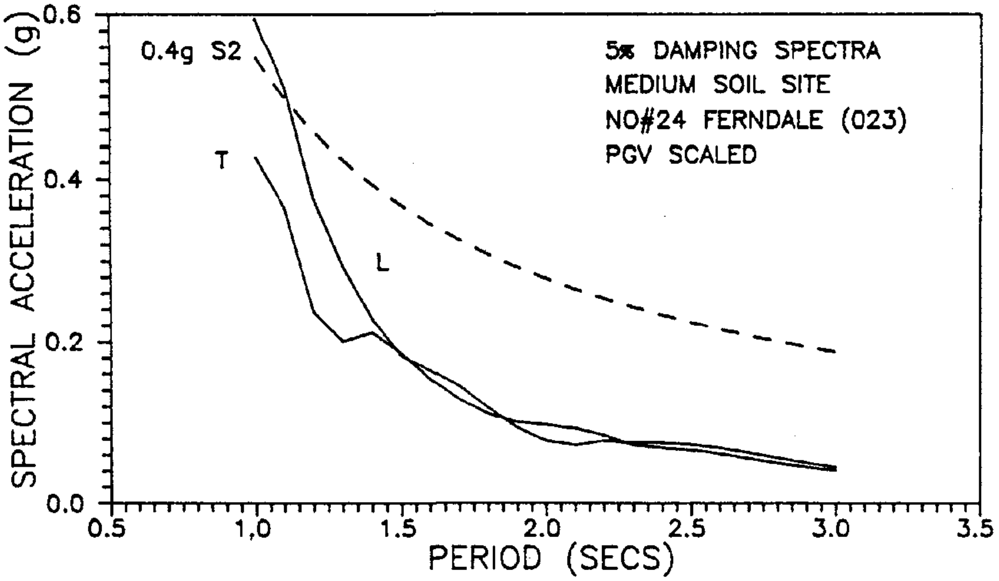
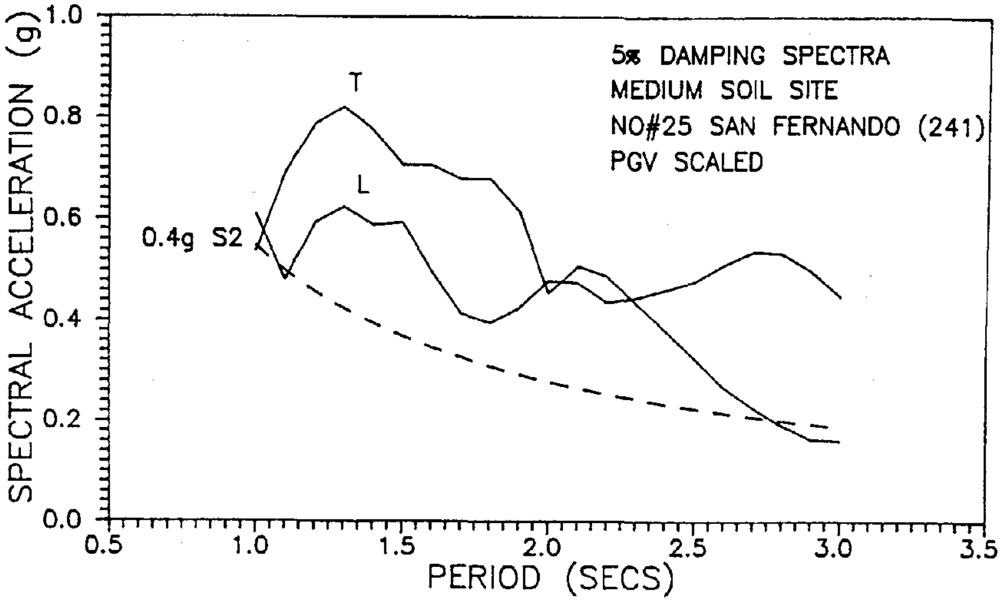


FIGURE B-1 Continued.



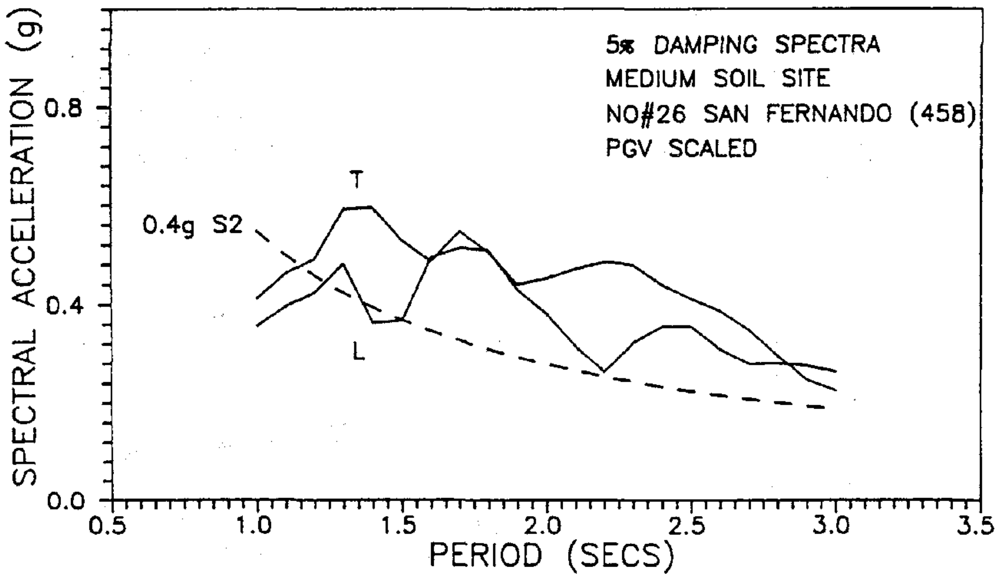
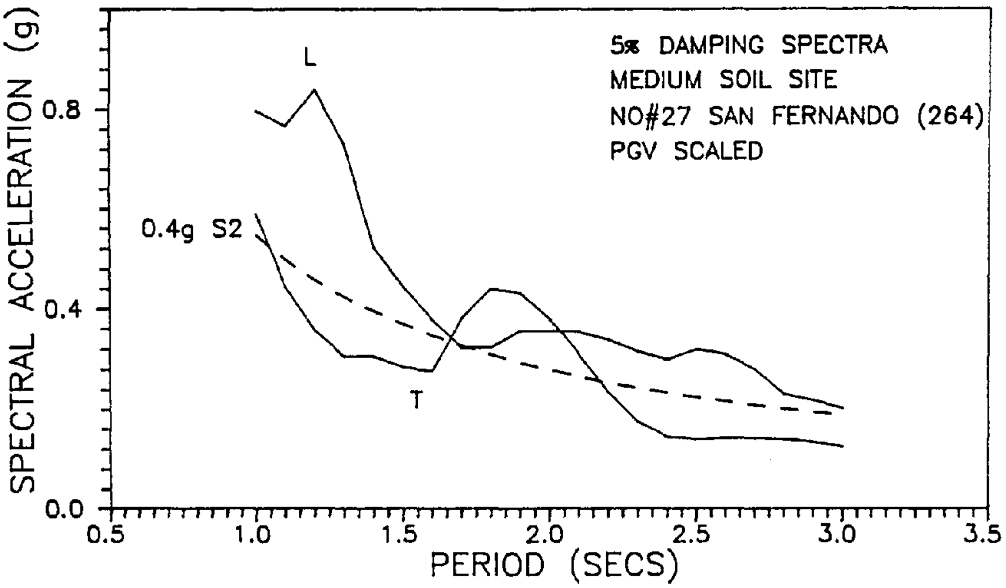


FIGURE B-1 Continued.



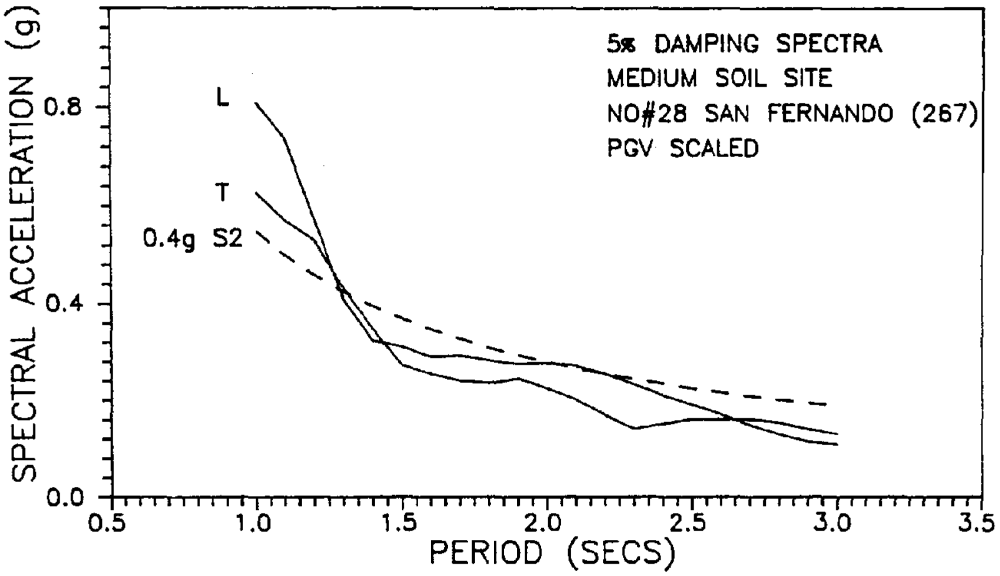


FIGURE B-1 Continued.

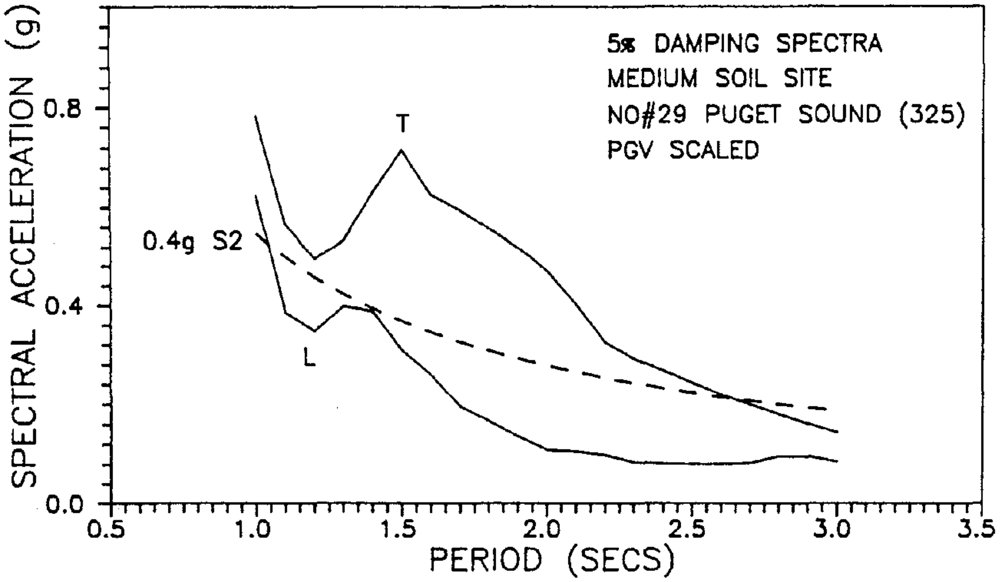


FIGURE B-1 Continued.

APPENDIX C

RESULTS OF MAXIMUM STORY SHEAR AND INTERSTORY DRIFT FOR 8 - STORY ISOLATED STRUCTURE. EXCITATION REPRESENTED BY A SET OF PAIRS OF SCALED EARTHQUAKE MOTIONS (SCALING BASED ON PGV).

TABLE C.1 Analysis results of maximum story shear for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Weight is total weight of structure including base.

EXCITATION		*2nd STORY SHEAR / WEIGHT*		*3rd STORY SHEAR / WEIGHT*		*4th STORY SHEAR / WEIGHT*		*5th STORY SHEAR / WEIGHT*		*6th STORY SHEAR / WEIGHT*		*7th STORY SHEAR / WEIGHT*		*8th STORY SHEAR / WEIGHT*	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
1	HELENA (323)	0.154	0.057	0.135	0.054	0.124	0.043	0.122	0.042	0.115	0.055	0.102	0.054	0.064	0.040
2	KERN COUNTY (095)	0.110	0.100	0.108	0.101	0.093	0.107	0.097	0.116	0.095	0.109	0.077	0.091	0.052	0.062
3	LYTLE CREEK (290)	0.102	0.096	0.081	0.080	0.112	0.102	0.135	0.103	0.121	0.088	0.126	0.115	0.082	0.093
4	PARKFIELD (097)	0.083	0.110	0.082	0.101	0.079	0.094	0.079	0.091	0.067	0.092	0.080	0.098	0.059	0.068
5	SAN FERNANDO (284)	0.107	0.118	0.102	0.116	0.094	0.114	0.082	0.102	0.069	0.095	0.063	0.081	0.043	0.050
6	SAN FERNANDO (126)	0.149	0.094	0.141	0.093	0.118	0.080	0.113	0.083	0.081	0.071	0.083	0.080	0.063	0.077
7	SAN FERNANDO (279)	0.096	0.114	0.092	0.100	0.083	0.092	0.083	0.087	0.079	0.078	0.073	0.079	0.056	0.056
8	SAN FERNANDO (104)	0.076	0.109	0.080	0.118	0.080	0.108	0.073	0.089	0.061	0.092	0.055	0.065	0.063	0.066
9	SAN FERNANDO (128)	0.094	0.119	0.089	0.111	0.084	0.105	0.106	0.101	0.090	0.097	0.083	0.096	0.092	0.095
10	SAN FERNANDO (220)	0.111	0.101	0.103	0.097	0.095	0.086	0.091	0.073	0.078	0.064	0.074	0.063	0.051	0.051

MEAN	0.108	0.102	0.101	0.097	0.096	0.093	0.098	0.089	0.086	0.084	0.082	0.082	0.063	0.066
σ	0.024	0.017	0.021	0.018	0.015	0.020	0.019	0.019	0.019	0.016	0.019	0.018	0.014	0.017
MEAN OF MAX (L,T)	0.120		0.111		0.107		0.105		0.096		0.089		0.068	
σ OF MAX(L,T)	0.017		0.017		0.010		0.015		0.014		0.016		0.015	

TABLE C.2 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Height is 12ft.

	EXCITATION	*2nd STORY DRIFT*/*HEIGHT* (%)		*3rd STORY DRIFT*/*HEIGHT* (%)		*4th STORY DRIFT*/*HEIGHT* (%)		*5th STORY DRIFT*/*HEIGHT* (%)		*6th STORY DRIFT*/*HEIGHT* (%)		*7th STORY DRIFT*/*HEIGHT* (%)		*8th STORY DRIFT*/*HEIGHT* (%)	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
1	HELENA (323)	0.3635	0.1325	0.3095	0.1294	0.3864	0.1406	0.3797	0.1344	0.3573	0.1767	0.4774	0.2563	0.2992	0.1909
2	KERN COUNTY (095)	0.2592	0.2397	0.2482	0.2446	0.2993	0.3408	0.3049	0.3674	0.2991	0.3449	0.3662	0.4264	0.2461	0.2934
3	LYTLE CREEK (290)	0.2404	0.2168	0.1808	0.1837	0.3485	0.3329	0.4148	0.3309	0.3706	0.2773	0.6003	0.5413	0.3824	0.4389
4	PARKFIELD (097)	0.1982	0.2477	0.1925	0.2297	0.2449	0.2918	0.2487	0.2837	0.2105	0.2945	0.3791	0.4628	0.2743	0.3302
5	SAN FERNANDO (284)	0.2506	0.2738	0.2394	0.2713	0.2954	0.3576	0.2589	0.3227	0.2145	0.3012	0.2966	0.3899	0.2009	0.2408
6	SAN FERNANDO (126)	0.3309	0.2207	0.3282	0.2214	0.3721	0.2585	0.3584	0.2537	0.2586	0.2311	0.3895	0.3763	0.2967	0.3628
7	SAN FERNANDO (279)	0.2312	0.2779	0.2171	0.2384	0.2625	0.2893	0.2597	0.2845	0.2491	0.2444	0.3393	0.3726	0.2581	0.2643
8	SAN FERNANDO (104)	0.1748	0.2253	0.1885	0.2837	0.2503	0.3441	0.2273	0.2800	0.1922	0.2910	0.2588	0.2958	0.2925	0.3130
9	SAN FERNANDO (128)	0.1969	0.2756	0.2099	0.2624	0.2681	0.3168	0.3252	0.3198	0.2857	0.3040	0.3861	0.4578	0.4278	0.4596
10	SAN FERNANDO (220)	0.2593	0.2390	0.2432	0.2295	0.2948	0.2712	0.2930	0.2317	0.2489	0.2017	0.3477	0.2978	0.2400	0.2428

MEAN	0.2505	0.2349	0.2357	0.2294	0.3022	0.2944	0.3071	0.2809	0.2887	0.2667	0.3841	0.3877	0.2918	0.3137
σ	0.0559	0.0404	0.0473	0.0427	0.0479	0.0601	0.0586	0.0615	0.0570	0.0493	0.0908	0.0835	0.0642	0.0824
MEAN OF MAX (L,T)	0.2754		0.2598		0.3342		0.3309		0.3020		0.4220		0.3245	
σ OF MAX(L,T)	0.0398		0.0393		0.0327		0.0447		0.0416		0.0799		0.0720	

TABLE C.3 Analysis results of maximum story shear for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Weight is total weight of structure including base.

	EXCITATION	2nd STORY SHEAR / WEIGHT		3rd STORY SHEAR / WEIGHT		4th STORY SHEAR / WEIGHT		5th STORY SHEAR / WEIGHT		6th STORY SHEAR / WEIGHT		7th STORY SHEAR / WEIGHT		8th STORY SHEAR / WEIGHT	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
11	LOWER CA (117)	0.132	0.113	0.116	0.096	0.101	0.137	0.092	0.141	0.099	0.118	0.096	0.106	0.069	0.080
12	EL CENTRO (117)	0.132	0.126	0.134	0.122	0.136	0.126	0.122	0.126	0.114	0.124	0.100	0.124	0.060	0.074
13	PARKFIELD (014)	0.084	0.133	0.082	0.121	0.101	0.111	0.098	0.097	0.105	0.099	0.113	0.120	0.075	0.078
14	SAN FERNANDO (110)	0.150	0.112	0.143	0.110	0.131	0.100	0.117	0.091	0.111	0.095	0.107	0.094	0.065	0.073
15	SAN FERNANDO (135)	0.099	0.165	0.091	0.151	0.072	0.129	0.084	0.103	0.089	0.108	0.079	0.105	0.058	0.085
16	SAN FERNANDO (208)	0.119	0.122	0.116	0.116	0.106	0.109	0.098	0.108	0.083	0.091	0.075	0.075	0.052	0.054
17	SAN FERNANDO (211)	0.121	0.127	0.116	0.119	0.109	0.108	0.100	0.093	0.087	0.082	0.076	0.075	0.050	0.052
18	SAN FERNANDO (466)	0.174	0.149	0.163	0.145	0.148	0.142	0.126	0.128	0.098	0.105	0.087	0.097	0.058	0.070
19	SAN FERNANDO (253)	0.100	0.129	0.096	0.117	0.083	0.102	0.095	0.100	0.080	0.095	0.064	0.104	0.058	0.080
20	SAN FERNANDO (199)	0.125	0.115	0.125	0.111	0.110	0.106	0.105	0.105	0.095	0.090	0.089	0.076	0.058	0.059

MEAN	0.124	0.129	0.118	0.121	0.110	0.117	0.104	0.109	0.096	0.101	0.089	0.098	0.060	0.071
σ	0.025	0.016	0.023	0.015	0.022	0.014	0.013	0.016	0.011	0.012	0.015	0.017	0.007	0.011
MEAN OF MAX (L,T)	0.139		0.131		0.122		0.113		0.104		0.100		0.070	
σ OF MAX(L,T)	0.017		0.016		0.015		0.014		0.011		0.016		0.011	

TABLE C.4 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Height is 12ft.

EXCITATION		$\frac{\text{2nd STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{3rd STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{4th STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{5th STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{6th STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{7th STORY DRIFT}}{\text{HEIGHT}}$(%)		$\frac{\text{8th STORY DRIFT}}{\text{HEIGHT}}$(%)	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
11	LOWER CA (117)	0.3095	0.2625	0.2700	0.2302	0.3270	0.4386	0.2844	0.4474	0.3051	0.3758	0.4480	0.5096	0.3219	0.3807
12	EL CENTRO (117)	0.3087	0.3007	0.3153	0.2893	0.4273	0.4175	0.3873	0.4046	0.3481	0.3844	0.4681	0.5934	0.2791	0.3549
13	PARKFIELD (014)	0.1896	0.2981	0.1894	0.2849	0.3169	0.3504	0.3043	0.3065	0.3318	0.3160	0.5363	0.5690	0.3529	0.3774
14	SAN FERNANDO (110)	0.3465	0.2514	0.3343	0.2637	0.4145	0.3304	0.3700	0.2872	0.3415	0.2978	0.5033	0.4481	0.3082	0.3477
15	SAN FERNANDO (135)	0.2276	0.3923	0.2154	0.3564	0.2326	0.4053	0.2649	0.3223	0.2759	0.3488	0.3699	0.4967	0.2755	0.4061
16	SAN FERNANDO (208)	0.2792	0.2745	0.2737	0.2708	0.3312	0.3394	0.3013	0.3394	0.2534	0.2887	0.3531	0.3575	0.2473	0.2529
17	SAN FERNANDO (211)	0.2867	0.2880	0.2739	0.2785	0.3442	0.3389	0.3136	0.2942	0.2695	0.2630	0.3497	0.3590	0.2322	0.2479
18	SAN FERNANDO (466)	0.4100	0.3554	0.3823	0.3425	0.4618	0.4476	0.3950	0.4050	0.3071	0.3359	0.4076	0.4634	0.2755	0.3346
19	SAN FERNANDO (253)	0.2390	0.3065	0.2265	0.2771	0.2661	0.3170	0.3010	0.3140	0.2544	0.2908	0.2995	0.4947	0.2733	0.3874
20	SAN FERNANDO (199)	0.2889	0.2678	0.2871	0.2614	0.3456	0.3374	0.3313	0.3308	0.2997	0.2854	0.4158	0.3493	0.2743	0.2752

MEAN	0.2886	0.2998	0.2768	0.2855	0.3467	0.3723	0.3253	0.3451	0.2987	0.3183	0.4151	0.4641	0.2840	0.3365
σ	0.0592	0.0415	0.0550	0.0357	0.0674	0.0468	0.0422	0.0517	0.0330	0.0386	0.0707	0.0822	0.0334	0.0549
MEAN OF MAX (L,T)	0.3228		0.3060		0.3844		0.3554		0.3263		0.4762		0.3365	
σ OF MAX(L,T)	0.0430		0.0374		0.0479		0.0462		0.0362		0.0752		0.0549	

TABLE C.5 Analysis results of maximum story shear for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Weight is total weight of structure including base.

EXCITATION		*2nd STORY SHEAR / WEIGHT*		*3rd STORY SHEAR / WEIGHT*		*4th STORY SHEAR / WEIGHT*		*5th STORY SHEAR / WEIGHT*		*6th STORY SHEAR / WEIGHT*		*7th STORY SHEAR / WEIGHT*		*8th STORY SHEAR / WEIGHT*	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
21	WESTERN WASH (325)	0.180	0.170	0.187	0.143	0.170	0.141	0.130	0.136	0.121	0.113	0.124	0.103	0.088	0.072
22	EUREKA (022)	0.135	0.172	0.151	0.143	0.152	0.134	0.132	0.123	0.121	0.104	0.110	0.092	0.063	0.076
23	EUREKA (023)	0.296	0.204	0.277	0.185	0.250	0.168	0.216	0.150	0.173	0.125	0.121	0.092	0.062	0.049
24	FERNDALE (023)	0.130	0.137	0.134	0.137	0.132	0.137	0.129	0.120	0.123	0.100	0.106	0.082	0.070	0.081
25	SAN FERNANDO (241)	0.217	0.200	0.201	0.173	0.181	0.159	0.154	0.139	0.121	0.125	0.093	0.101	0.053	0.073
26	SAN FERNANDO (458)	0.215	0.175	0.202	0.167	0.187	0.153	0.165	0.135	0.136	0.110	0.104	0.080	0.057	0.048
27	SAN FERNANDO (264)	0.232	0.124	0.212	0.113	0.178	0.108	0.156	0.108	0.134	0.100	0.111	0.107	0.087	0.086
28	SAN FERNANDO (267)	0.164	0.173	0.145	0.146	0.130	0.126	0.125	0.118	0.100	0.131	0.097	0.121	0.078	0.102
29	PUGET SOUND (325)	0.132	0.256	0.132	0.222	0.128	0.189	0.128	0.159	0.106	0.132	0.093	0.129	0.056	0.110

MEAN	0.189	0.179	0.160	0.159	0.168	0.146	0.148	0.132	0.126	0.115	0.107	0.101	0.068	0.077
σ	0.053	0.036	0.042	0.030	0.036	0.023	0.028	0.015	0.020	0.012	0.011	0.016	0.013	0.020
MEAN OF MAX (L,T)	0.209		0.193		0.175		0.152		0.133		0.114		0.082	
σ OF MAX(L,T)	0.046		0.042		0.033		0.026		0.015		0.009		0.016	

TABLE C.6 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 39.132 in (T_b = 2 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Height is 12ft.

	EXCITATION	$\frac{\text{2nd STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{3rd STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{4th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{5th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{6th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{7th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{8th STORY DRIFT}}{\text{HEIGHT}}$ (%)	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
21	WESTERN WASH (325)	0.4167	0.4008	0.4349	0.3400	0.5351	0.4466	0.4077	0.4268	0.3740	0.3569	0.5636	0.4875	0.4122	0.3410
22	EUREKA (022)	0.3058	0.3953	0.3481	0.3386	0.4720	0.4254	0.4094	0.3862	0.3801	0.3236	0.5202	0.4241	0.3021	0.3624
23	EUREKA (023)	0.6922	0.4776	0.6489	0.4371	0.7848	0.5369	0.6823	0.4812	0.5464	0.4013	0.5699	0.4464	0.2897	0.2392
24	FERNDALE (023)	0.3034	0.3040	0.3153	0.3236	0.4181	0.4339	0.3967	0.3803	0.3814	0.3150	0.4987	0.3724	0.3287	0.3776
25	SAN FERNANDO (241)	0.5059	0.4746	0.4698	0.4101	0.5625	0.5090	0.4831	0.4487	0.3826	0.4033	0.4489	0.4881	0.2533	0.3468
26	SAN FERNADO (458)	0.4986	0.4129	0.4722	0.3947	0.5885	0.4805	0.5218	0.4293	0.4297	0.3507	0.4914	0.3845	0.2724	0.2264
27	SAN FERNANDO (264)	0.5511	0.2944	0.5023	0.2762	0.5594	0.3444	0.4850	0.3282	0.4204	0.3150	0.5164	0.5115	0.4059	0.4078
28	SAN FERNANDO (267)	0.3918	0.4073	0.3438	0.3451	0.4077	0.4024	0.3842	0.3743	0.3047	0.4165	0.4471	0.5759	0.3700	0.4903
29	PUGET SOUND (325)	0.3100	0.6007	0.3097	0.5308	0.4018	0.6298	0.3995	0.4992	0.3258	0.4091	0.4413	0.5967	0.2638	0.5202

MEAN	0.4417	0.4186	0.4272	0.3774	0.5255	0.4677	0.4633	0.4171	0.3939	0.3657	0.4997	0.4763	0.3220	0.3680
σ	0.1248	0.0878	0.1045	0.0711	0.1142	0.0787	0.0901	0.0518	0.0656	0.0399	0.0454	0.0736	0.0573	0.0929
1MEAN OF MAX (L,T)	0.4858		0.4529		0.5526		0.4765		0.4179		0.5357		0.3866	
σ OF MAX(L,T)	0.1120		0.0984		0.1071		0.0862		0.0490		0.0387		0.0777	

TABLE C.7 **Analysis results of maximum story shear for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Weight is total weight of structure including base.**

	EXCITATION	*2nd STORY SHEAR / WEIGHT*		*3rd STORY SHEAR / WEIGHT*		*4th STORY SHEAR / WEIGHT*		*5th STORY SHEAR / WEIGHT*		*6th STORY SHEAR / WEIGHT*		*7th STORY SHEAR / WEIGHT*		*8th STORY SHEAR / WEIGHT*	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
1	HELENA (323)	0.079	0.035	0.079	0.037	0.076	0.038	0.075	0.033	0.068	0.039	0.063	0.038	0.041	0.031
2	KERN COUNTY (095)	0.065	0.068	0.064	0.061	0.058	0.057	0.057	0.054	0.056	0.049	0.059	0.053	0.035	0.040
3	LYTLE CREEK (290)	0.078	0.059	0.078	0.044	0.084	0.060	0.087	0.062	0.076	0.060	0.076	0.068	0.049	0.051
4	PARKFIELD (097)	0.048	0.064	0.048	0.055	0.046	0.046	0.043	0.048	0.037	0.052	0.040	0.058	0.030	0.043
5	SAN FERNANDO (284)	0.056	0.068	0.056	0.067	0.054	0.061	0.049	0.057	0.043	0.053	0.049	0.054	0.028	0.038
6	SAN FERNANDO (126)	0.075	0.052	0.068	0.055	0.064	0.053	0.074	0.053	0.061	0.057	0.065	0.057	0.037	0.045
7	SAN FERNANDO (279)	0.048	0.061	0.043	0.060	0.043	0.057	0.046	0.054	0.048	0.045	0.045	0.050	0.041	0.039
8	SAN FERNANDO (104)	0.054	0.0077	0.049	0.082	0.050	0.069	0.048	0.056	0.045	0.057	0.043	0.044	0.044	0.043
9	SAN FERNANDO (128)	0.072	0.077	0.065	0.067	0.055	0.059	0.054	0.068	0.058	0.061	0.053	0.060	0.051	0.058
10	SAN FERNANDO (220)	0.073	0.059	0.066	0.058	0.061	0.058	0.065	0.053	0.058	0.046	0.054	0.046	0.044	0.039

MEAN	0.065	0.062	0.062	0.059	0.059	0.055	0.060	0.054	0.055	0.052	0.055	0.053	0.040	0.043
σ	0.012	0.012	0.012	0.012	0.012	0.010	0.014	0.009	0.011	0.007	0.011	0.008	0.007	0.007
MEAN OF MAX (L,T)	0.072		0.069		0.064		0.064		0.059		0.058		0.045	
σ OF MAX(L,T)	0.006		0.008		0.010		0.011		0.008		0.008		0.006	

TABLE C.8 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Height is 12ft.

	EXCITATION	$\frac{\text{2nd STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{3rd STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{4th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{5th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{6th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{7th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{8th STORY DRIFT}}{\text{HEIGHT}}$ (%)	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
1	HELENA (323)	0.1821	0.0798	0.1837	0.0887	0.2368	0.0940	0.2355	0.1071	0.2124	0.1228	0.2927	0.1795	0.1905	0.1467
2	KERN COUNTY (095)	0.1490	0.1583	0.1469	0.1447	0.1842	0.1792	0.1771	0.1674	0.1766	0.1542	0.2788	0.2524	0.1677	0.1907
3	LYTLE CREEK (290)	0.1725	0.1315	0.1815	0.1037	0.2750	0.1963	0.2810	0.1988	0.2386	0.1881	0.3582	0.3217	0.2275	0.2417
4	PARKFIELD (097)	0.1093	0.1481	0.1117	0.1327	0.1445	0.1468	0.1366	0.1503	0.1174	0.1679	0.1930	0.2795	0.1384	0.2035
5	SAN FERNANDO (284)	0.1305	0.1558	0.1324	0.1575	0.1707	0.1952	0.1551	0.1745	0.1315	0.1667	0.2261	0.2584	0.1288	0.1811
6	SAN FERNANDO (126)	0.1701	0.1206	0.1561	0.1300	0.1982	0.1746	0.2343	0.1670	0.1910	0.1804	0.3015	0.2735	0.1734	0.2137
7	SAN FERNANDO (279)	0.1178	0.1494	0.1000	0.1408	0.1287	0.1807	0.1467	0.1715	0.1531	0.1454	0.2155	0.2392	0.1894	0.1861
8	SAN FERNANDO (104)	0.1167	0.1610	0.1163	0.1962	0.1573	0.2221	0.1518	0.1756	0.1408	0.1799	0.2011	0.2103	0.2053	0.2070
9	SAN FERNANDO (128)	0.1591	0.1713	0.1517	0.1579	0.1721	0.1811	0.1702	0.2138	0.1843	0.1930	0.2458	0.2883	0.2346	0.2812
10	SAN FERNANDO (220)	0.1644	0.1406	0.1513	0.1348	0.1944	0.1829	0.2026	0.1672	0.1823	0.1459	0.2530	0.2166	0.2069	0.1854

MEAN	0.1471	0.1416	0.1432	0.1387	0.1862	0.1753	0.1891	0.1693	0.1728	0.1644	0.2566	0.2519	0.1863	0.2037
σ	0.0252	0.0249	0.0267	0.0281	0.0412	0.0325	0.0452	0.0268	0.0356	0.0211	0.0490	0.0398	0.0331	0.0348
MEAN OF MAX (L,T)	0.1633		0.1605		0.2015		0.2016		0.1862		0.2760		0.2106	
σ OF MAX(L,T)	0.0103		0.0193		0.0336		0.0377		0.0234		0.0379		0.0286	

TABLE C.9 **Analysis results of maximum story shear for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Weight is total weight of structure including base.**

	EXCITATION	*2nd STORY SHEAR / WEIGHT*		*3rd STORY SHEAR / WEIGHT*		*4th STORY SHEAR / WEIGHT*		*5th STORY SHEAR / WEIGHT*		*6th STORY SHEAR / WEIGHT*		*7th STORY SHEAR / WEIGHT*		*8th STORY SHEAR / WEIGHT*	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
11	LOWER CA (117)	0.071	0.074	0.069	0.070	0.067	0.072	0.061	0.068	0.061	0.062	0.054	0.063	0.041	0.057
12	EL CENTRO (117)	0.108	0.083	0.106	0.085	0.098	0.079	0.084	0.091	0.074	0.081	0.063	0.083	0.041	0.052
13	PARKFIELD (014)	0.058	0.079	0.059	0.070	0.073	0.064	0.067	0.060	0.072	0.059	0.068	0.069	0.050	0.057
14	SAN FERNANDO (110)	0.090	0.075	0.083	0.070	0.076	0.074	0.065	0.073	0.062	0.058	0.059	0.049	0.041	0.044
15	SAN FERNANDO (135)	0.094	0.149	0.089	0.131	0.078	0.112	0.067	0.091	0.060	0.070	0.057	0.057	0.037	0.046
16	SAN FERNANDO (208)	0.076	0.093	0.072	0.087	0.067	0.080	0.061	0.072	0.055	0.061	0.044	0.048	0.031	0.033
17	SAN FERNANDO (211)	0.087	0.091	0.080	0.084	0.074	0.076	0.068	0.066	0.058	0.053	0.044	0.046	0.029	0.035
18	SAN FERNANDO (466)	0.117	0.144	0.114	0.128	0.109	0.112	0.098	0.095	0.080	0.076	0.058	0.065	0.038	0.048
19	SAN FERNANDO (253)	0.066	0.157	0.060	0.142	0.059	0.127	0.054	0.107	0.049	0.083	0.044	0.072	0.043	0.054
20	SAN FERNANDO (199)	0.089	0.078	0.092	0.075	0.082	0.071	0.070	0.079	0.071	0.067	0.067	0.056	0.046	0.042

MEAN	0.086	0.102	0.082	0.094	0.078	0.087	0.070	0.080	0.064	0.067	0.056	0.061	0.040	0.047
σ	0.017	0.032	0.017	0.027	0.014	0.021	0.012	0.014	0.009	0.010	0.009	0.011	0.006	0.008
MEAN OF MAX (L,T)	0.107		0.099		0.091		0.081		0.070		0.063		0.047	
σ OF MAX(L,T)	0.029		0.025		0.019		0.014		0.009		0.011		0.008	

TABLE C.10 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Height is 12ft.

	EXCITATION	*2nd STORY DRIFT / HEIGHT* (%)		*3rd STORY DRIFT / HEIGHT* (%)		*4th STORY DRIFT / HEIGHT* (%)		*5th STORY DRIFT / HEIGHT* (%)		*6th STORY DRIFT / HEIGHT* (%)		*7th STORY DRIFT / HEIGHT* (%)		*8th STORY DRIFT / HEIGHT* (%)	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
11	LOWER CA (117)	0.1638	0.1603	0.1609	0.1630	0.2133	0.2353	0.1923	0.2191	0.1880	0.1933	0.2522	0.3031	0.1901	0.2706
12	EL CENTRO (117)	0.2552	0.1935	0.2479	0.1973	0.3049	0.2436	0.2616	0.2803	0.2278	0.2552	0.2911	0.3952	0.1895	0.2465
13	PARKFIELD (014)	0.1287	0.1794	0.1386	0.1657	0.2309	0.2057	0.2094	0.1902	0.2257	0.1875	0.3209	0.3297	0.2345	0.2757
14	SAN FERNANDO (110)	0.2081	0.1669	0.1942	0.1676	0.2376	0.2290	0.2039	0.2278	0.1938	0.1841	0.2725	0.2383	0.1915	0.2102
15	SAN FERNANDO (135)	0.2167	0.3590	0.2069	0.3106	0.2450	0.3490	0.2090	0.2877	0.1853	0.2201	0.2667	0.2720	0.1763	0.2220
16	SAN FERNANDO (208)	0.1819	0.2227	0.1689	0.2045	0.2087	0.2501	0.1921	0.2255	0.1721	0.1943	0.2078	0.2300	0.1445	0.1592
17	SAN FERNANDO (211)	0.2100	0.2128	0.1872	0.1967	0.2288	0.2387	0.2120	0.2081	0.1803	0.1699	0.2037	0.2144	0.1360	0.1645
18	SAN FERNANDO (466)	0.2726	0.3450	0.2667	0.3050	0.3395	0.3519	0.3055	0.3013	0.2505	0.2414	0.2728	0.3080	0.1761	0.2289
19	SAN FERNANDO (253)	0.1553	0.3768	0.1406	0.3367	0.1838	0.3972	0.1693	0.3392	0.1496	0.2631	0.2087	0.3410	0.2049	0.2594
20	SAN FERNANDO (199)	0.2077	0.1843	0.2163	0.1708	0.2607	0.2314	0.2211	0.2498	0.2197	0.2123	0.3072	0.2586	0.2166	0.1977

MEAN	0.2000	0.2401	0.1928	0.2218	0.2453	0.2732	0.2176	0.2529	0.1993	0.2121	0.2604	0.2890	0.1860	0.2235
σ	0.0418	0.0810	0.0407	0.0646	0.0440	0.0629	0.0370	0.0450	0.0291	0.0304	0.0398	0.0539	0.0285	0.0391
MEAN OF MAX (L,T)	0.2531		0.2341		0.2856		0.2556		0.2196		0.2973		0.2254	
σ OF MAX(L,T)	0.0742		0.0596		0.0576		0.0427		0.0277		0.0504		0.0382	

TABLE C.11 **Analysis results of maximum story shear for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Weight is total weight of structure including base.**

	EXCITATION	*2nd STORY SHEAR / WEIGHT*		*3rd STORY SHEAR / WEIGHT*		*4th STORY SHEAR / WEIGHT*		*5th STORY SHEAR / WEIGHT*		*6th STORY SHEAR / WEIGHT*		*7th STORY SHEAR / WEIGHT*		*8th STORY SHEAR / WEIGHT*	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
21	WESTERN WASH (325)	0.101	0.133	0.093	0.126	0.082	0.115	0.069	0.094	0.060	0.083	0.057	0.067	0.043	0.040
22	EUREKA (022)	0.100	0.109	0.088	0.099	0.072	0.089	0.067	0.075	0.060	0.065	0.045	0.062	0.025	0.046
23	EUREKA (023)	0.174	0.088	0.160	0.079	0.141	0.076	0.119	0.065	0.093	0.054	0.064	0.045	0.036	0.028
24	FERNDALE (023)	0.067	0.077	0.066	0.077	0.064	0.074	0.062	0.065	0.061	0.055	0.054	0.051	0.037	0.047
25	SAN FERNANDO (241)	0.204	0.151	0.180	0.143	0.154	0.127	0.125	0.105	0.103	0.088	0.077	0.068	0.042	0.045
26	SAN FERNANDO (458)	0.161	0.164	0.137	0.140	0.113	0.129	0.093	0.110	0.073	0.087	0.054	0.064	0.030	0.034
27	SAN FERNANDO (264)	0.111	0.085	0.106	0.079	0.093	0.074	0.084	0.068	0.070	0.066	0.081	0.066	0.051	0.061
28	SAN FERNANDO (267)	0.080	0.107	0.072	0.099	0.082	0.076	0.081	0.075	0.059	0.084	0.054	0.086	0.048	0.056
29	PUGET SOUND (325)	0.075	0.132	0.078	0.118	0.086	0.110	0.084	0.088	0.078	0.081	0.065	0.067	0.050	0.056

MEAN	0.119	0.116	0.109	0.107	0.099	0.097	0.087	0.083	0.073	0.074	0.061	0.064	0.040	0.046
σ	0.046	0.029	0.038	0.025	0.029	0.022	0.021	0.016	0.015	0.013	0.011	0.011	0.009	0.011
MEAN OF MAX (L,T)	0.135		0.123		0.110		0.093		0.081		0.069		0.047	
σ OF MAX(L,T)	0.037		0.031		0.026		0.019		0.013		0.010		0.009	

TABLE C.12 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.05. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Height is 12ft.

	EXCITATION	$\frac{\textit{2nd STORY DRIFT}}{\textit{HEIGHT}}$(%)		$\frac{\textit{3rd STORY DRIFT}}{\textit{HEIGHT}}$(%)		$\frac{\textit{4th STORY DRIFT}}{\textit{HEIGHT}}$(%)		$\frac{\textit{5th STORY DRIFT}}{\textit{HEIGHT}}$(%)		$\frac{\textit{6th STORY DRIFT}}{\textit{HEIGHT}}$(%)		$\frac{\textit{7th STORY DRIFT}}{\textit{HEIGHT}}$(%)		$\frac{\textit{8th STORY DRIFT}}{\textit{HEIGHT}}$(%)	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
21	WESTERN WASH (325)	0.2361	0.3137	0.2174	0.2995	0.2559	0.3637	0.2157	0.3019	0.1926	0.2619	0.2660	0.3179	0.2013	0.1854
22	EUREKA (022)	0.2319	0.2502	0.2063	0.2317	0.2257	0.2771	0.2115	0.2362	0.1898	0.2063	0.2085	0.2896	0.1186	0.2202
23	EUREKA (023)	0.4099	0.1966	0.3757	0.1893	0.4440	0.2414	0.3731	0.2098	0.2918	0.1730	0.3007	0.2124	0.1681	0.1335
24	FERNDALE (023)	0.1532	0.1681	0.1509	0.1821	0.2016	0.2365	0.1939	0.2039	0.1889	0.1735	0.2552	0.2313	0.1752	0.2216
25	SAN FERNANDO (241)	0.4851	0.3522	0.4238	0.3428	0.4804	0.4127	0.3949	0.3302	0.3206	0.2792	0.3599	0.3283	0.2004	0.2176
26	SAN FERNADO (458)	0.3864	0.3920	0.3222	0.3344	0.3488	0.4126	0.2939	0.3526	0.2294	0.2818	0.2549	0.3071	0.1433	0.1622
27	SAN FERNANDO (264)	0.2558	0.2007	0.2483	0.1862	0.2909	0.2316	0.2654	0.2091	0.2223	0.2023	0.3850	0.3163	0.2416	0.2856
28	SAN FERNANDO (267)	0.1909	0.2498	0.1702	0.2373	0.2497	0.2430	0.2492	0.2328	0.1811	0.2706	0.2571	0.4048	0.2281	0.2638
29	PUGET SOUND (325)	0.1669	0.2958	0.1864	0.2865	0.2661	0.3646	0.2604	0.2840	0.2428	0.2558	0.2990	0.3160	0.2307	0.2652

MEAN	0.2796	0.2688	0.2557	0.2544	0.3070	0.3092	0.2731	0.2623	0.2288	0.2338	0.2874	0.3026	0.1897	0.2172
σ	0.1115	0.0712	0.0908	0.0598	0.0920	0.0736	0.0662	0.0531	0.0463	0.0423	0.0525	0.0528	0.0394	0.0474
MEAN OF MAX (L,T)	0.3134		0.2910		0.3466		0.2957		0.2556		0.3262		0.2228	
σ OF MAX(L,T)	0.0929		0.0722		0.0829		0.0617		0.0401		0.0451		0.0403	

TABLE C.13 Analysis results of maximum story shear for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Weight is total weight of structure including base.

	EXCITATION	*2nd STORY SHEAR* / *WEIGHT*		*3rd STORY SHEAR* / *WEIGHT*		*4th STORY SHEAR* / *WEIGHT*		*5th STORY SHEAR* / *WEIGHT*		*6th STORY SHEAR* / *WEIGHT*		*7th STORY SHEAR* / *WEIGHT*		*8th STORY SHEAR* / *WEIGHT*	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
1	HELENA (323)	0.135	0.057	0.137	0.055	0.124	0.047	0.121	0.042	0.112	0.054	0.100	0.054	0.061	0.041
2	KERN COUNTY (095)	0.108	0.094	0.108	0.093	0.092	0.101	0.099	0.110	0.097	0.104	0.080	0.091	0.052	0.063
3	LYTLE CREEK (290)	0.102	0.096	0.080	0.079	0.112	0.100	0.135	0.101	0.120	0.087	0.126	0.115	0.082	0.092
4	PARKFIELD (097)	0.084	0.123	0.084	0.104	0.079	0.096	0.072	0.093	0.061	0.097	0.076	0.098	0.057	0.065
5	SAN FERNANDO (284)	0.104	0.119	0.099	0.121	0.091	0.116	0.080	0.101	0.069	0.092	0.063	0.079	0.043	0.049
6	SAN FERNANDO (126)	0.151	0.088	0.142	0.089	0.118	0.077	0.116	0.091	0.081	0.074	0.083	0.080	0.065	0.077
7	SAN FERNANDO (279)	0.096	0.097	0.073	0.095	0.072	0.090	0.086	0.083	0.079	0.075	0.072	0.074	0.055	0.056
8	SAN FERNANDO (104)	0.075	0.109	0.080	0.117	0.080	0.109	0.072	0.091	0.060	0.092	0.055	0.065	0.062	0.066
9	SAN FERNANDO (128)	0.092	0.119	0.087	0.111	0.085	0.106	0.107	0.100	0.089	0.096	0.082	0.094	0.092	0.095
10	SAN FERNANDO (220)	0.105	0.099	0.097	0.094	0.095	0.081	0.092	0.072	0.078	0.063	0.074	0.064	0.051	0.051

MEAN	0.105	0.100	0.099	0.096	0.095	0.092	0.098	0.088	0.085	0.083	0.082	0.081	0.062	0.066
σ	0.022	0.018	0.023	0.018	0.017	0.019	0.020	0.018	0.019	0.015	0.019	0.017	0.014	0.017
MEAN OF MAX (L,T)	0.117		0.111		0.107		0.105		0.095		0.088		0.068	
σ OF MAX(L,T)	0.016		0.018		0.011		0.015		0.013		0.017		0.015	

TABLE C.14 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Rock Sites (representative of soil type S1). Height is 12ft.

	EXCITATION	2nd STORY DRIFT / HEIGHT (%)		3rd STORY DRIFT / HEIGHT (%)		4th STORY DRIFT / HEIGHT (%)		5th STORY DRIFT / HEIGHT (%)		6th STORY DRIFT / HEIGHT (%)		7th STORY DRIFT / HEIGHT (%)		8th STORY DRIFT / HEIGHT (%)	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
1	HELENA (323)	0.2996	0.1327	0.3149	0.1300	0.3838	0.1489	0.3777	0.1312	0.3479	0.1743	0.4722	0.2550	0.2922	0.1947
2	KERN COUNTY (095)	0.2394	0.2204	0.2479	0.2277	0.2930	0.3231	0.3072	0.3472	0.3014	0.3297	0.3804	0.4283	0.2468	0.2973
3	LYTLE CREEK (290)	0.2407	0.2144	0.1802	0.1819	0.3486	0.3277	0.4142	0.3270	0.3694	0.2750	0.6003	0.5400	0.3820	0.4338
4	PARKFIELD (097)	0.1964	0.2723	0.1958	0.2389	0.2473	0.3018	0.2267	0.2905	0.1913	0.3088	0.3629	0.4663	0.2670	0.3170
5	SAN FERNANDO (284)	0.2436	0.2776	0.2332	0.2814	0.2876	0.3653	0.2518	0.3208	0.2146	0.2955	0.2964	0.3812	0.2005	0.2382
6	SAN FERNANDO (126)	0.3359	0.2078	0.3296	0.2123	0.3713	0.2468	0.3676	0.2797	0.2598	0.2350	0.3889	0.3801	0.3066	0.3674
7	SAN FERNANDO (279)	0.2164	0.2279	0.1727	0.2229	0.2223	0.2847	0.2667	0.2603	0.2475	0.2390	0.3375	0.3496	0.2557	0.2625
8	SAN FERNANDO (104)	0.1743	2244	0.1887	0.2822	0.2508	0.3455	0.2265	0.2920	0.1846	0.2911	0.2556	0.2979	0.2905	0.3140
9	SAN FERNANDO (128)	0.2016	0.2686	0.2047	0.2608	0.2656	0.3205	0.3295	0.3191	0.2838	0.3018	0.3835	0.4479	0.4312	0.4564
10	SAN FERNANDO (220)	0.2457	0.2327	0.2293	0.2221	0.2948	0.2574	0.2912	0.2280	0.2490	0.1995	0.3510	0.3056	0.2399	0.2451

MEAN	0.2394	0.2279	0.2297	0.2260	0.2966	0.2922	0.3059	0.2796	0.2649	0.2650	0.3829	0.3852	0.2912	0.3126
σ	0.0459	0.0398	0.0517	0.0436	0.0518	0.0593	0.0621	0.0595	0.0587	0.0482	0.0908	0.0828	0.0654	0.0807
MEAN OF MAX (L,T)	0.2632		0.2590		0.3339		0.3297		0.3001		0.4184		0.3224	
σ OF MAX(L,T)	0.0333		0.0422		0.0324		0.0444		0.0389		0.0808		0.0712	

TABLE C.15 Analysis results of maximum story shear for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Weight is total weight of structure including base.

EXCITATION		*2nd STORY SHEAR / WEIGHT*		*3rd STORY SHEAR / WEIGHT*		*4th STORY SHEAR / WEIGHT*		*5th STORY SHEAR / WEIGHT*		*6th STORY SHEAR / WEIGHT*		*7th STORY SHEAR / WEIGHT*		*8th STORY SHEAR / WEIGHT*	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
11	LOWER CA (117)	0.114	0.115	0.103	0.100	0.092	0.139	0.095	0.141	0.093	0.117	0.084	0.107	0.062	0.079
12	EL CENTRO (117)	0.116	0.126	0.127	0.114	0.126	0.119	0.113	0.135	0.111	0.128	0.097	0.124	0.058	0.074
13	PARKFIELD (014)	0.081	0.140	0.078	0.126	0.105	0.113	0.101	0.099	0.102	0.103	0.109	0.127	0.080	0.079
14	SAN FERNANDO (110)	0.121	0.094	0.116	0.094	0.105	0.098	0.097	0.097	0.097	0.100	0.090	0.088	0.060	0.064
15	SAN FERNANDO (135)	0.110	0.131	0.101	0.133	0.080	0.122	0.086	0.113	0.093	0.107	0.081	0.104	0.059	0.077
16	SAN FERNANDO (208)	0.111	0.122	0.108	0.117	0.103	0.110	0.094	0.109	0.082	0.090	0.075	0.074	0.052	0.050
17	SAN FERNANDO (211)	0.120	0.123	0.116	0.116	0.108	0.106	0.097	0.091	0.084	0.080	0.073	0.072	0.051	0.051
18	SAN FERNANDO (466)	0.110	0.118	0.108	0.111	0.102	0.102	0.089	0.090	0.079	0.084	0.087	0.097	0.058	0.069
19	SAN FERNANDO (253)	0.098	0.114	0.096	0.098	0.083	0.103	0.096	0.101	0.080	0.094	0.064	0.104	0.057	0.080
20	SAN FERNANDO (199)	0.133	0.116	0.135	0.114	0.117	0.105	0.102	0.105	0.103	0.090	0.095	0.075	0.057	0.058

MEAN	0.111	0.120	0.109	0.112	0.102	0.112	0.097	0.108	0.092	0.099	0.086	0.097	0.059	0.068
σ	0.013	0.012	0.015	0.012	0.013	0.012	0.007	0.016	0.010	0.014	0.012	0.019	0.008	0.011
MEAN OF MAX (L,T)	0.124		0.118		0.115		0.109		0.101		0.100		0.068	
σ OF MAX(L,T)	0.008		0.012		0.011		0.016		0.013		0.017		0.011	

TABLE C.16 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Stiff Soil Sites (representative of soil type S1). Height is 12ft.

	EXCITATION	$\frac{\text{2nd STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{3rd STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{4th STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{5th STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{6th STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{7th STORY DRIFT}}{\text{HEIGHT}}(\%)$		$\frac{\text{8th STORY DRIFT}}{\text{HEIGHT}}(\%)$	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
11	LOWER CA (117)	0.2673	0.2661	0.2421	0.2313	0.2843	0.4457	0.2969	0.4462	0.2907	0.3715	0.3959	0.5148	0.2875	0.3795
12	EL CENTRO (117)	0.2714	0.2865	0.2993	0.2788	0.3996	0.3866	0.3556	0.4135	0.3412	0.3977	0.4542	0.5923	0.2725	0.3529
13	PARKFIELD (014)	0.1840	0.3136	0.1815	0.2973	0.3274	0.3570	0.3143	0.3108	0.3220	0.3272	0.5168	0.6036	0.3724	0.3831
14	SAN FERNANDO (110)	0.2779	0.2185	0.2712	0.2230	0.3311	0.3087	0.2994	0.3073	0.2982	0.3141	0.4227	0.4256	0.2835	0.3053
15	SAN FERNANDO (135)	0.2536	0.3076	0.2362	0.3087	0.2529	0.3839	0.2729	0.3578	0.2879	0.3421	0.3802	0.4928	0.2810	0.3677
16	SAN FERNANDO (208)	0.2591	0.2762	0.2541	0.2721	0.3290	0.3455	0.2912	0.3440	0.2561	0.2867	0.3504	0.3508	0.2467	0.2357
17	SAN FERNANDO (211)	0.2751	0.2781	0.2724	0.2707	0.3437	0.3335	0.3004	0.2896	0.2637	0.2572	0.3405	0.3465	0.2370	0.2420
18	SAN FERNANDO (466)	0.2564	0.2846	0.2530	0.2630	0.3202	0.3230	0.2804	0.2859	0.2533	0.2677	0.4084	0.4636	0.2752	0.3307
19	SAN FERNANDO (253)	0.2338	0.2595	0.2258	0.2267	0.2668	0.3204	0.3020	0.3172	0.2526	0.2893	0.2970	0.4938	0.2710	0.3869
20	SAN FERNANDO (199)	0.2999	0.2606	0.3114	0.2598	0.3656	0.3346	0.3184	0.3306	0.3279	0.2849	0.4476	0.3489	0.2674	0.2745

MEAN	0.2579	0.2751	0.2547	0.2631	0.3221	0.3539	0.3032	0.3403	0.2894	0.3138	0.4014	0.4633	0.2794	0.3258
σ	0.0297	0.0255	0.0353	0.0275	0.0422	0.0392	0.0218	0.0500	0.0312	0.0434	0.0604	0.0903	0.0345	0.0553
MEAN OF MAX (L,T)	0.2851		0.2764		0.3616		0.3405		0.3188		0.4731		0.3269	
σ OF MAX(L,T)	0.0164		0.0267		0.0371		0.0492		0.0416		0.0823		0.0536	

TABLE C.17 Analysis results of maximum story shear for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2).

	EXCITATION	*2nd STORY SHEAR / WEIGHT*		*3rd STORY SHEAR / WEIGHT*		*4th STORY SHEAR / WEIGHT*		*5th STORY SHEAR / WEIGHT*		*6th STORY SHEAR / WEIGHT*		*7th STORY SHEAR / WEIGHT*		*8th STORY SHEAR / WEIGHT*	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
21	WESTERN WASH (325)	0.148	0.134	0.156	0.123	0.150	0.115	0.124	0.123	0.119	0.118	0.118	0.104	0.082	0075
22	EUREKA (022)	0.132	0.146	0.140	0.121	0.137	0.113	0.116	0.108	0.099	0.103	0.089	0.103	0.050	0.074
23	EUREKA (023)	0.154	0.108	0.141	0.097	0.130	0.100	0.114	0.097	0.091	0.091	0.074	0.073	0.051	0.045
24	FERNDALE (023)	0.125	0.126	0.131	0.122	0.129	0.121	0.133	0.109	0.125	0.093	0.106	0.081	0.068	0.076
25	SAN FERNANDO (241)	0.140	0.146	0.135	0.139	0.124	0.127	0.107	0.123	0.088	0.111	0.073	0.108	0.050	0.081
26	SAN FERNANDO (458)	0.125	0.156	0.121	0.146	0.120	0.136	0.116	0.120	0.096	0.099	0.074	0.081	0.048	0.056
27	SAN FERNANDO (264)	0.162	0.132	0.163	0.116	0.157	0.109	0.150	0.122	0.139	0.110	0.122	0.106	0.079	0.087
28	SAN FERNANDO (267)	0.120	0.174	0.120	0.140	0.124	0.114	0.125	0.107	0.097	0.105	0.086	0.125	0.074	0.094
29	PUGET SOUND (325)	0.126	0.164	0.132	0.160	0.134	0.152	0.135	0.134	0.122	0.131	0.110	0.131	0.074	0.113

MEAN	0.137	0.143	0.138	0.129	0.134	0.121	0.124	0.116	0.108	0.107	0.095	0.101	0.064	0.078
σ	0.014	0.019	0.014	0.018	0.012	0.015	0.012	0.011	0.017	0.012	0.019	0.019	0.013	0.019
MEAN OF MAX (L,T)	0.153		0.146		0.138		0.127		0.114		0.108		0.079	
σ OF MAX(L,T)	0.013		0.010		0.011		0.011		0.015		0.018		0.018	

TABLE C.18 Analysis results of maximum interstory drift for 8 - story isolated structure with R = 88.048 in (T_b = 3 sec.), f_{max} = 0.10. Excitation represented by a set of pairs of scaled earthquake motions recorded on Medium Soil Sites (representative of soil type S2). Height is 12ft.

	EXCITATION	*2nd STORY DRIFT* / *HEIGHT* (%)		*3rd STORY DRIFT* / *HEIGHT* (%)		*4th STORY DRIFT* / *HEIGHT* (%)		*5th STORY DRIFT* / *HEIGHT* (%)		*6th STORY DRIFT* / *HEIGHT* (%)		*7th STORY DRIFT* / *HEIGHT* (%)		*8th STORY DRIFT* / *HEIGHT* (%)	
		L	T	L	T	L	T	L	T	L	T	L	T	L	T
21	WESTERN WASH (325)	0.3429	0.3176	0.3663	0.2884	0.4703	0.3709	0.3885	0.3915	0.3721	0.3749	0.5415	0.4919	0.3833	0.3557
22	EUREKA (022)	0.3025	0.3381	0.3253	0.2859	0.4289	0.3620	0.3613	0.3388	0.3156	0.3210	0.4225	0.4801	0.2365	0.3569
23	EUREKA (023)	0.3621	0.2516	0.3327	0.2290	0.4194	0.3145	0.3597	0.3046	0.2856	0.2805	0.3484	0.3567	0.2365	0.2125
24	FERNDALE (023)	0.2794	0.2751	0.2963	0.2893	0.3960	0.3844	0.4108	0.3432	0.3890	0.2949	0.5016	0.3708	0.3216	0.3589
25	SAN FERNANDO (241)	0.3241	0.3351	0.3133	0.3258	0.3854	0.4031	0.3363	0.3896	0.2788	0.3526	0.3457	0.5095	0.2353	0.3856
26	SAN FERNADO (458)	0.2832	0.3675	0.2826	0.3451	0.3778	0.4267	0.3604	0.3789	0.3024	0.3158	0.3545	0.3820	0.2245	0.2640
27	SAN FERNANDO (264)	0.3752	0.2936	0.3760	0.2655	0.4868	0.3412	0.4739	0.3693	0.4353	0.3352	0.5739	0.5024	0.3703	0.4113
28	SAN FERNANDO (267)	0.2844	0.3952	0.2817	0.3320	0.3775	0.3644	0.3847	0.3351	0.2971	0.3369	0.4127	0.5978	0.3499	0.4591
29	PUGET SOUND (325)	0.2820	0.3859	0.3125	0.3779	0.4202	0.4806	0.4200	0.4201	0.3773	0.4071	0.5055	0.6099	0.3443	0.5373

MEAN	0.3151	0.3289	0.3207	0.3043	0.4180	0.3831	0.3884	0.3635	0.3392	0.3354	0.4451	0.4779	0.3002	0.3713
σ	0.0352	0.0435	0.0316	0.0426	0.0371	0.0462	0.0392	0.0337	0.0521	0.0369	0.0828	0.0875	0.0621	0.0909
MEAN OF MAX (L,T)	0.3535		0.3419		0.4321		0.3967		0.3576		0.5059		0.3770	
σ OF MAX(L,T)	0.0328		0.0255		0.0367		0.0331		0.0453		0.0840		0.0863	

APPENDIX D

RESULTS OF MAXIMUM STORY SHEAR AND INTERSTORY DRIFT FOR 8 - STORY ISOLATED STRUCTURE. EXCITATION REPRESENTED BY SCALED PAIRS OF REAL RECORDS ACCORDING TO THE DYNAMIC ANALYSIS PROCEDURE OF SEAOC.

TABLE D.1 Analysis results of maximum story shear for 8 - story isolated structure with R=39.132 in (T_b = 2 sec), f_{max} = 0.10, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Weight is total weight of structure including base.

EXCITATION	*2nd STORY SHEAR* / *WEIGHT*		*3rd STORY SHEAR* / *WEIGHT*		*4th STORY SHEAR* / *WEIGHT*		*5th STORY SHEAR* / *WEIGHT*		*6th STORY SHEAR* / *WEIGHT*		*7th STORY SHEAR* / *WEIGHT*		*8th STORY SHEAR* / *WEIGHT*	
	L	T	L	T	L	T	L	T	L	T	L	T	L	T
MOTION #1	0.113	0.142	0.115	0.129	0.113	0.112	0.103	0.109	0.097	0.092	0.082	0.088	0.050	0.066
MOTION #2	0.109	0.125	0.106	0.120	0.095	0.113	0.086	0.112	0.074	0.098	0.067	0.082	0.044	0.058
MOTION #3	0.109	0.129	0.109	0.126	0.111	0.128	0.100	0.130	0.093	0.126	0.078	0.124	0.045	0.074
MOTION #4	0.126	0.183	0.120	0.184	0.108	0.163	0.101	0.130	0.090	0.125	0.087	0.121	0.061	0.086
MOTION #5	0.124	0.211	0.108	0.189	0.102	0.159	0.107	0.142	0.098	0.129	0.107	0.112	0.083	0.082
MOTION #6	0.143	0.193	0.158	0.171	0.159	0.160	0.139	0.140	0.129	0.116	0.118	0.099	0.065	0.087

TABLE D.2 Analysis results of maximum interstory drift for 8 - story isolated structure with R=39.132 in (T_b = 2 sec), f_{max} = 0.10, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Height is 12ft.

EXCITATION	$\frac{\text{2nd STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{3rd STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{4th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{5th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{6th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{7th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{8th STORY DRIFT}}{\text{HEIGHT}}$ (%)	
MOTION #1	0.2571	0.3422	0.2642	0.3087	0.3504	0.3626	0.3207	0.3469	0.3025	0.2883	0.3887	0.4640	0.2388	0.3097
MOTION #2	0.2563	0.2801	0.2492	0.2789	0.2972	0.3514	0.2637	0.3534	0.2348	0.3101	0.3143	0.3902	0.2054	0.2753
MOTION #3	0.2543	0.3022	0.2571	0.3007	0.3512	0.4238	0.3166	0.4059	0.2869	0.3909	0.3675	0.5920	0.2092	0.3546
MOTION #4	0.2986	0.4283	0.2846	0.4350	0.3374	0.5195	0.3126	0.4114	0.2843	0.3912	0.4041	0.5609	0.2833	0.4089
MOTION #5	0.2812	0.5055	0.2620	0.4529	0.3277	0.5031	0.3235	0.4510	0.2965	0.4098	0.5047	0.5303	0.3916	0.3884
MOTION #6	0.3206	0.4430	0.3629	0.4034	0.4927	0.5114	0.4330	0.4358	0.4052	0.3568	0.5585	0.4576	0.3123	0.4142

TABLE D.3 Analysis results of maximum story shear for 8 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.05, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Weight is total weight of structure including base.

EXCITATION	*2nd STORY SHEAR / WEIGHT*		*3rd STORY SHEAR / WEIGHT*		*4th STORY SHEAR / WEIGHT*		*5th STORY SHEAR / WEIGHT*		*6th STORY SHEAR / WEIGHT*		*7th STORY SHEAR / WEIGHT*		*8th STORY SHEAR / WEIGHT*	
	L	T	L	T	L	T	L	T	L	T	L	T	L	T
MOTION #1	0.064	0.080	0.062	0.082	0.057	0.078	0.053	0.077	0.053	0.060	0.051	0.066	0.036	0.039
MOTION #2	0.057	0.104	0.056	0.096	0.058	0.088	0.055	0.078	0.049	0.066	0.038	0.052	0.026	0.037
MOTION #3	0.085	0.083	0.087	0.091	0.082	0.081	0.072	0.093	0.066	0.082	0.057	0.085	0.036	0.054
MOTION #4	0.096	0.123	0.089	0.111	0.080	0.0102	0.068	0.082	0.060	0.059	0.051	0.065	0.040	0.045
MOTION #5	0.079	0.104	0.076	0.102	0.068	0.091	0.067	0.084	0.063	0.067	0.065	0.075	0.058	0.046
MOTION #6	0.120	0.130	0.106	0.115	0.092	0.103	0.084	0.087	0.071	0.077	0.053	0.068	0.030	0.048

TABLE D.4 Analysis results of maximum interstory drift for 8 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.05, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Height is 12ft.

EXCITATION	$\frac{\text{2nd STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{3rd STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{4th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{5th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{6th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{7th STORY DRIFT}}{\text{HEIGHT}}$ (%)		$\frac{\text{8th STORY DRIFT}}{\text{HEIGHT}}$ (%)	
MOTION #1	0.1411	0.1915	0.1446	0.1909	0.1801	0.2439	0.1634	0.2436	0.1679	0.1903	0.2389	0.3087	0.1713	0.1802
MOTION #2	0.1349	0.2443	0.1335	0.2253	0.1854	0.2744	0.1727	0.2455	0.1523	0.2088	0.1816	0.2466	0.1215	0.1772
MOTION #3	0.2012	0.1903	0.2024	0.2108	0.2552	0.2495	0.2255	0.2867	0.2025	0.2597	0.2664	0.4028	0.1688	0.2568
MOTION #4	0.2236	0.2856	0.2093	0.2610	0.2504	0.3150	0.2164	0.2549	0.1896	0.1934	0.2412	0.3089	0.1867	0.2124
MOTION #5	0.1796	0.2421	0.1760	0.2418	0.2109	0.2877	0.2028	0.2668	0.1922	0.2168	0.3050	0.3611	0.2718	0.2168
MOTION #6	0.2837	0.3111	0.2476	0.2702	0.2872	0.3237	0.2648	0.2743	0.2243	0.2460	0.2491	0.3207	0.1437	0.2294

TABLE D.5 Analysis results of maximum story shear for 8 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.10, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Weight is total weight of structure including base.

EXCITATION	$\frac{\text{2nd STORY SHEAR}}{\text{WEIGHT}}$		$\frac{\text{3rd STORY SHEAR}}{\text{WEIGHT}}$		$\frac{\text{4th STORY SHEAR}}{\text{WEIGHT}}$		$\frac{\text{5th STORY SHEAR}}{\text{WEIGHT}}$		$\frac{\text{6th STORY SHEAR}}{\text{WEIGHT}}$		$\frac{\text{7th STORY SHEAR}}{\text{WEIGHT}}$		$\frac{\text{8th STORY SHEAR}}{\text{WEIGHT}}$	
	L	T	L	T	L	T	L	T	L	T	L	T	L	T
MOTION #1	0.115	0.118	0.117	0.117	0.114	0.112	0.101	0.108	0.095	0.091	0.081	0.098	0.050	0.065
MOTION #2	0.103	0.129	0.099	0.124	0.093	0.117	0.085	0.116	0.073	0.098	0.066	0.083	0.043	0.056
MOTION #3	0.112	0.132	0.111	0.118	0.107	0.121	0.096	0.136	0.090	0.128	0.075	0.122	0.044	0.073
MOTION #4	0.115	0.152	0.104	0.157	0.108	0.150	0.100	0.125	0.093	0.122	0.085	0.119	0.065	0.082
MOTION #5	0.131	0.150	0.111	0.148	0.105	0.145	0.116	0.135	0.104	0.131	0.106	0.118	0.085	0.075
MOTION #6	0.140	0.153	0.141	0.132	0.129	0.113	0.102	0.102	0.094	0.099	0.077	0.115	0.046	0.084

TABLE D.6 Analysis results of maximum interstory drift for 8 - story isolated structure with R=88.048 in (T_b = 3 sec), f_{max} = 0.10, excited by scaled pairs of real records according to the Dynamic Analysis Procedure of SEAOC. Height is 12ft.

EXCITATION	$\frac{\textit{2nd STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{3rd STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{4th STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{5th STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{6th STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{7th STORY DRIFT}}{\textit{HEIGHT}}$ (%)		$\frac{\textit{8th STORY DRIFT}}{\textit{HEIGHT}}$ (%)	
MOTION #1	0.2638	0.2754	0.2711	0.2845	0.3539	0.3608	0.3166	0.3433	0.2977	0.2857	0.3817	0.4636	0.2371	0.3060
MOTION #2	0.2405	0.2867	0.2338	0.2877	0.2890	0.3682	0.2611	0.3691	0.2326	0.3123	0.3121	0.3953	0.2038	0.2661
MOTION #3	0.2517	0.2960	0.2569	0.2855	0.3326	0.3991	0.2991	0.4181	0.2784	0.3979	0.3550	0.5861	0.2046	0.3504
MOTION #4	0.2726	0.3556	0.2428	0.3725	0.3361	0.4745	0.3071	0.3959	0.2923	0.3898	0.4037	0.5495	0.3013	0.3868
MOTION #5	0.2892	0.3526	0.2513	0.3516	0.3248	0.4495	0.3499	0.4283	0.3159	0.4126	0.4980	0.5611	0.3979	0.3523
MOTION #6	0.3217	0.3537	0.3283	0.3110	0.4046	0.3602	0.3203	0.3170	0.3013	0.3057	0.3686	0.5356	0.2144	0.4028

APPENDIX E

CONVERSION TO SI UNITS

To convert	To	Multiply by
in.	mm	25.4
ft	mm	304.8
kip	kN	4.459
psf	Pa	47.88
ksi	MPa	6.895

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
LIST OF 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). This report is available only through NTIS (see address given above).
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).
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). This report is available only through NTIS (see address given above).

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, (PB90-162348/AS).

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, (PB89-174429/AS).
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, (PB89-174437/AS).
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, (PB89-163737/AS).
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, (PB89-207153/AS).
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, (PB89-207146/AS).
NCEER-88-0037	"Optimal Placement of Actuators for Structural Control," by F.Y. Cheng and C.P. Pantelides, 8/15/88, (PB89-162846/AS).
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, (PB89-218457/AS).
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, (PB90-145681/AS).
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, (PB89-189625/AS).
NCEER-88-0042	"Modeling Strong Ground Motion from Multiple Event Earthquakes," by G.W. Ellis and A.S. Cakmak, 10/15/88, (PB89-174445/AS).
NCEER-88-0043	"Nonstationary Models of Seismic Ground Acceleration," by M. Grigoriu, S.E. Ruiz and E. Rosenblueth, 7/15/88, (PB89-189617/AS).
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, (PB89-174452/AS).
NCEER-88-0045	"First Expert Panel Meeting on Disaster Research and Planning," edited by J. Pantelic and J. Stoyle, 9/15/88, (PB89-174460/AS).
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, (PB89-208383/AS).

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, (PB89-174478/AS).
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, (PB89-207179/AS).
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, (PB89-207187/AS).
NCEER-89-0003	"Hysteretic Columns Under Random Excitation," by G-Q. Cai and Y.K. Lin, 1/9/89, (PB89-196513/AS).
NCEER-89-0004	"Experimental Study of 'Elephant Foot Bulge' Instability of Thin-Walled Metal Tanks," by Z-H. Jia and R.L. Ketter, 2/22/89, (PB89-207195/AS).
NCEER-89-0005	"Experiment on Performance of Buried Pipelines Across San Andreas Fault," by J. Isenberg, E. Richardson and T.D. O'Rourke, 3/10/89, (PB89-218440/AS).
NCEER-89-0006	"A Knowledge-Based Approach to Structural Design of Earthquake-Resistant Buildings," by M. Subramani, P. Gergely, C.H. Conley, J.F. Abel and A.H. Zaghw, 1/15/89, (PB89-218465/AS).
NCEER-89-0007	"Liquefaction Hazards and Their Effects on Buried Pipelines," by T.D. O'Rourke and P.A. Lane, 2/1/89, (PB89-218481).
NCEER-89-0008	"Fundamentals of System Identification in Structural Dynamics," by H. Imai, C-B. Yun, O. Maruyama and M. Shinozuka, 1/26/89, (PB89-207211/AS).
NCEER-89-0009	"Effects of the 1985 Michoacan Earthquake on Water Systems and Other Buried Lifelines in Mexico," by A.G. Ayala and M.J. O'Rourke, 3/8/89, (PB89-207229/AS).
NCEER-89-R010	"NCEER Bibliography of Earthquake Education Materials," by K.E.K. Ross, Second Revision, 9/1/89, (PB90-125352/AS).
NCEER-89-0011	"Inelastic Three-Dimensional Response Analysis of Reinforced Concrete Building Structures (IDARC-3D), Part I - Modeling," by S.K. Kunnath and A.M. Reinhorn, 4/17/89, (PB90-114612/AS).
NCEER-89-0012	"Recommended Modifications to ATC-14," by C.D. Poland and J.O. Malley, 4/12/89, (PB90-108648/AS).
NCEER-89-0013	"Repair and Strengthening of Beam-to-Column Connections Subjected to Earthquake Loading," by M. Corazao and A.J. Durrani, 2/28/89, (PB90-109885/AS).
NCEER-89-0014	"Program EXKAL2 for Identification of Structural Dynamic Systems," by O. Maruyama, C-B. Yun, M. Hoshiya and M. Shinozuka, 5/19/89, (PB90-109877/AS).
NCEER-89-0015	"Response of Frames With Bolted Semi-Rigid Connections, Part I - Experimental Study and Analytical Predictions," by P.J. DiCorso, A.M. Reinhorn, J.R. Dickerson, J.B. Radziminski and W.L. Harper, 6/1/89, to be published.
NCEER-89-0016	"ARMA Monte Carlo Simulation in Probabilistic Structural Analysis," by P.D. Spanos and M.P. Mignolet, 7/10/89, (PB90-109893/AS).
NCEER-89-P017	"Preliminary Proceedings from the Conference on Disaster Preparedness - The Place of Earthquake Education in Our Schools," Edited by K.E.K. Ross, 6/23/89.
NCEER-89-0017	"Proceedings from the Conference on Disaster Preparedness - The Place of Earthquake Education in Our Schools," Edited by K.E.K. Ross, 12/31/89, (PB90-207895).

NCEER-89-0018	"Multidimensional Models of Hysteretic Material Behavior for Vibration Analysis of Shape Memory Energy Absorbing Devices, by E.J. Graesser and F.A. Cozzarelli, 6/7/89, (PB90-164146/AS).
NCEER-89-0019	"Nonlinear Dynamic Analysis of Three-Dimensional Base Isolated Structures (3D-BASIS)," by S. Nagarajaiah, A.M. Reinhorn and M.C. Constantinou, 8/3/89, (PB90-161936/AS).
NCEER-89-0020	"Structural Control Considering Time-Rate of Control Forces and Control Rate Constraints," by F.Y. Cheng and C.P. Pantelides, 8/3/89, (PB90-120445/AS).
NCEER-89-0021	"Subsurface Conditions of Memphis and Shelby County," by K.W. Ng, T-S. Chang and H-H.M. Hwang, 7/26/89, (PB90-120437/AS).
NCEER-89-0022	"Seismic Wave Propagation Effects on Straight Jointed Buried Pipelines," by K. Elhmadi and M.J. O'Rourke, 8/24/89, (PB90-162322/AS).
NCEER-89-0023	"Workshop on Serviceability Analysis of Water Delivery Systems," edited by M. Grigoriu, 3/6/89, (PB90-127424/AS).
NCEER-89-0024	"Shaking Table Study of a 1/5 Scale Steel Frame Composed of Tapered Members," by K.C. Chang, J.S. Hwang and G.C. Lee, 9/18/89, (PB90-160169/AS).
NCEER-89-0025	"DYNA1D: A Computer Program for Nonlinear Seismic Site Response Analysis - Technical Documentation," by Jean H. Prevost, 9/14/89, (PB90-161944/AS).
NCEER-89-0026	"1:4 Scale Model Studies of Active Tendon Systems and Active Mass Dampers for Aseismic Protection," by A.M. Reinhorn, T.T. Soong, R.C. Lin, Y.P. Yang, Y. Fukao, H. Abe and M. Nakai, 9/15/89, (PB90-173246/AS).
NCEER-89-0027	"Scattering of Waves by Inclusions in a Nonhomogeneous Elastic Half Space Solved by Boundary Element Methods," by P.K. Hadley, A. Askar and A.S. Cakmak, 6/15/89, (PB90-145699/AS).
NCEER-89-0028	"Statistical Evaluation of Deflection Amplification Factors for Reinforced Concrete Structures," by H.H.M. Hwang, J-W. Jaw and A.L. Ch'ng, 8/31/89, (PB90-164633/AS).
NCEER-89-0029	"Bedrock Accelerations in Memphis Area Due to Large New Madrid Earthquakes," by H.H.M. Hwang, C.H.S. Chen and G. Yu, 11/7/89, (PB90-162330/AS).
NCEER-89-0030	"Seismic Behavior and Response Sensitivity of Secondary Structural Systems," by Y.Q. Chen and T.T. Soong, 10/23/89, (PB90-164658/AS).
NCEER-89-0031	"Random Vibration and Reliability Analysis of Primary-Secondary Structural Systems," by Y. Ibrahim, M. Grigoriu and T.T. Soong, 11/10/89, (PB90-161951/AS).
NCEER-89-0032	"Proceedings from the Second U.S. - Japan Workshop on Liquefaction, Large Ground Deformation and Their Effects on Lifelines, September 26-29, 1989," Edited by T.D. O'Rourke and M. Hamada, 12/1/89, (PB90-209388/AS).
NCEER-89-0033	"Deterministic Model for Seismic Damage Evaluation of Reinforced Concrete Structures," by J.M. Bracci, A.M. Reinhorn, J.B. Mander and S.K. Kunnath, 9/27/89.
NCEER-89-0034	"On the Relation Between Local and Global Damage Indices," by E. DiPasquale and A.S. Cakmak, 8/15/89, (PB90-173865).
NCEER-89-0035	"Cyclic Undrained Behavior of Nonplastic and Low Plasticity Silts," by A.J. Walker and H.E. Stewart, 7/26/89, (PB90-183518/AS).
NCEER-89-0036	"Liquefaction Potential of Surficial Deposits in the City of Buffalo, New York," by M. Budhu, R. Giese and L. Baumgrass, 1/17/89, (PB90-208455/AS).

NCEER-89-0037	"A Determinstic Assessment of Effects of Ground Motion Incoherence," by A.S. Veletsos and Y. Tang, 7/15/89, (PB90-164294/AS).
NCEER-89-0038	"Workshop on Ground Motion Parameters for Seismic Hazard Mapping," July 17-18, 1989, edited by R.V. Whitman, 12/1/89, (PB90-173923/AS).
NCEER-89-0039	"Seismic Effects on Elevated Transit Lines of the New York City Transit Authority," by C.J. Costantino, C.A. Miller and E. Heymsfield, 12/26/89, (PB90-207887/AS).
NCEER-89-0040	"Centrifugal Modeling of Dynamic Soil-Structure Interaction," by K. Weissman, Supervised by J.H. Prevost, 5/10/89, (PB90-207879/AS).
NCEER-89-0041	"Linearized Identification of Buildings With Cores for Seismic Vulnerability Assessment," by I-K. Ho and A.E. Aktan, 11/1/89, (PB90-251943/AS).
NCEER-90-0001	"Geotechnical and Lifeline Aspects of the October 17, 1989 Loma Prieta Earthquake in San Francisco," by T.D. O'Rourke, H.E. Stewart, F.T. Blackburn and T.S. Dickerman, 1/90, (PB90-208596/AS).
NCEER-90-0002	"Nonnormal Secondary Response Due to Yielding in a Primary Structure," by D.C.K. Chen and L.D. Lutes, 2/28/90, (PB90-251976/AS).
NCEER-90-0003	"Earthquake Education Materials for Grades K-12," by K.E.K. Ross, 4/16/90, (PB91-113415/AS).
NCEER-90-0004	"Catalog of Strong Motion Stations in Eastern North America," by R.W. Busby, 4/3/90, (PB90-251984)/AS.
NCEER-90-0005	"NCEER Strong-Motion Data Base: A User Manuel for the GeoBase Release (Version 1.0 for the Sun3)," by P. Friberg and K. Jacob, 3/31/90 (PB90-258062/AS).
NCEER-90-0006	"Seismic Hazard Along a Crude Oil Pipeline in the Event of an 1811-1812 Type New Madrid Earthquake," by H.H.M. Hwang and C-H.S. Chen, 4/16/90(PB90-258054).
NCEER-90-0007	"Site-Specific Response Spectra for Memphis Sheahan Pumping Station," by H.H.M. Hwang and C.S. Lee, 5/15/90, (PB91-108811/AS).
NCEER-90-0008	"Pilot Study on Seismic Vulnerability of Crude Oil Transmission Systems," by T. Ariman, R. Dobry, M. Grigoriu, F. Kozin, M. O'Rourke, T. O'Rourke and M. Shinozuka, 5/25/90, (PB91-108837/AS).
NCEER-90-0009	"A Program to Generate Site Dependent Time Histories: EQGEN," by G.W. Ellis, M. Srinivasan and A.S. Cakmak, 1/30/90, (PB91-1108829/AS).
NCEER-90-0010	"Active Isolation for Seismic Protection of Operating Rooms," by M.E. Talbott, Supervised by M. Shinozuka, 6/8/9, (PB91-110205/AS).
NCEER-90-0011	"Program LINEARID for Identification of Linear Structural Dynamic Systems," by C-B. Yun and M. Shinozuka, 6/25/90, (PB91-110312/AS).
NCEER-90-0012	"Two-Dimensional Two-Phase Elasto-Plastic Seismic Response of Earth Dams," by A.N. Yiagos, Supervised by J.H. Prevost, 6/20/90, (PB91-110197/AS).
NCEER-90-0013	"Secondary Systems in Base-Isolated Structures: Experimental Investigation, Stochastic Response and Stochastic Sensitivity," by G.D. Manolis, G. Juhn, M.C. Constantinou and A.M. Reinhorn, 7/1/90, (PB91-110320/AS).
NCEER-90-0014	"Seismic Behavior of Lightly-Reinforced Concrete Column and Beam-Column Joint Details," by S.P. Pessiki, C.H. Conley, P. Gergely and R.N. White, 8/22/90, (PB91-108795/AS).
NCEER-90-0015	"Two Hybrid Control Systems for Building Structures Under Strong Earthquakes," by J.N. Yang and A. Danielians, 6/29/90, (PB91-125393/AS).

NCEER-90-0016	"Instantaneous Optimal Control with Acceleration and Velocity Feedback," by J.N. Yang and Z. Li, 6/29/90, (PB91-125401/AS).
NCEER-90-0017	"Reconnaissance Report on the Northern Iran Earthquake of June 21, 1990," by M. Mehrain, 10/4/90, (PB91-125377/AS).
NCEER-90-0018	"Evaluation of Liquefaction Potential in Memphis and Shelby County," by T.S. Chang, P.S. Tang, C.S. Lee and H. Hwang, 8/10/90, (PB91-125427/AS).
NCEER-90-0019	"Experimental and Analytical Study of a Combined Sliding Disc Bearing and Helical Steel Spring Isolation System," by M.C. Constantinou, A.S. Mokha and A.M. Reinhorn, 10/4/90, (PB91-125385/AS).
NCEER-90-0020	"Experimental Study and Analytical Prediction of Earthquake Response of a Sliding Isolation System with a Spherical Surface," by A.S. Mokha, M.C. Constantinou and A.M. Reinhorn, 10/11/90, (PB91-125419/AS).
NCEER-90-0021	"Dynamic Interaction Factors for Floating Pile Groups," by G. Gazetas, K. Fan, A. Kaynia and E. Kausel, 9/10/90, (PB91-170381/AS).
NCEER-90-0022	"Evaluation of Seismic Damage Indices for Reinforced Concrete Structures," by S. Rodríguez-Gómez and A.S. Cakmak, 9/30/90, PB91-171322/AS).
NCEER-90-0023	"Study of Site Response at a Selected Memphis Site," by H. Desai, S. Ahmad, E.S. Gazetas and M.R. Oh, 10/11/90.
NCEER-90-0024	"A User's Guide to *Strongmo: Version 1.0 of NCEER's Strong-Motion Data Access Tool for PCs and Terminals*," by P.A. Friberg and C.A.T. Susch, 11/15/90, (PB91-171272/AS).
NCEER-90-0025	"A Three-Dimensional Analytical Study of Spatial Variability of Seismic Ground Motions," by L-L. Hong and A.H.-S. Ang, 10/30/90, (PB91-170399/AS).
NCEER-90-0026	"MUMOID User's Guide - A Program for the Identification of Modal Parameters," by S. Rodríguez-Gómez and E. DiPasquale, 9/30/90, (PB91-171298/AS).
NCEER-90-0027	"SARCF-II User's Guide - Seismic Analysis of Reinforced Concrete Frames," by S. Rodríguez-Gómez, Y.S. Chung and C. Meyer, 9/30/90.
NCEER-90-0028	"Viscous Dampers: Testing, Modeling and Application in Vibration and Seismic Isolation," by N. Makris and M.C. Constantinou, 12/20/90 (PB91-190561/AS).
NCEER-90-0029	"Soil Effects on Earthquake Ground Motions in the Memphis Area," by H. Hwang, C.S. Lee, K.W. Ng and T.S. Chang, 8/2/90, (PB91-190751/AS).
NCEER-91-0001	"Proceedings from the Third Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction, December 17-19, 1990," edited by T.D. O'Rourke and M. Hamada, 2/1/91, (PB91-179259/AS).
NCEER-91-0002	"Physical Space Solutions of Non-Proportionally Damped Systems," by M. Tong, Z. Liang and G.C. Lee, 1/15/91, (PB91-179242/AS).
NCEER-91-0003	"Kinematic Seismic Response of Single Piles and Pile Groups," by K. Fan, G. Gazetas, A. Kaynia, E. Kausel and S. Ahmad, 1/10/91, to be published.
NCEER-91-0004	"Theory of Complex Damping," by Z. Liang and G. Lee, to be published.
NCEER-91-0005	"3D-BASIS - Nonlinear Dynamic Analysis of Three Dimensional Base Isolated Structures: Part II," by S. Nagarajaiah, A.M. Reinhorn and M.C. Constantinou, 2/28/91, (PB91-190553/AS).

NCEER-91-0006	"A Multidimensional Hysteretic Model for Plasticity Deforming Metals in Energy Absorbing Devices," by E.J. Graesser and F.A. Cozzarelli, 4/9/91.
NCEER-91-0007	"A Framework for Customizable Knowledge-Based Expert Systems with an Application to a KBES for Evaluating the Seismic Resistance of Existing Buildings," by E.G. Ibarra-Anaya and S.J. Fenves, 4/9/91.
NCEER-91-0008	"Nonlinear Analysis of Steel Frames with Semi-Rigid Connections Using the Capacity Spectrum Method," by G.G. Deierlein, S-H. Hsieh, Y-J. Shen and J.F. Abel, 7/2/91.
NCEER-91-0009	"Earthquake Education Materials for Grades K-12," by K.E.K. Ross, 4/30/91.
NCEER-91-0010	"Phase Wave Velocities and Displacement Phase Differences in a Harmonically Oscillating Pile," by N. Makris and G. Gazetas, 7/8/91.
NCEER-91-0011	"Dynamic Characteristics of a Full-Sized Five-Story Steel Structure and a 2/5 Model," by K.C. Chang, G.C. Yao, G.C. Lee, D.S. Hao and Y.C. Yeh," to be published.
NCEER-91-0012	"Seismic Response of a 2/5 Scale Steel Structure with Added Viscoelastic Dampers," by K.C. Chang, T.T. Soong, S-T. Oh and M.L. Lai, 5/17/91.
NCEER-91-0013	"Earthquake Response of Retaining Walls; Full-Scale Testing and Computational Modeling," by S. Alampalli and A-W.M. Elgamal, 6/20/91, to be published.
NCEER-91-0014	"3D-BASIS-M: Nonlinear Dynamic Analysis of Multiple Building Base Isolated Structures," by P.C. Tsopelas, S. Nagarajaiah, M.C. Constantinou and A.M. Reinhorn, 5/28/91.
NCEER-91-0015	"Evaluation of SEAOC Design Requirements for Sliding Isolated Structures," by D. Theodossiou and M.C. Constantinou, 6/10/91.