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TECHNICAL MANUAL

SEISMIC DESIGN GUIDELINES FOR ESSENTIAL BUILDINGS

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DEPARTMENTS OF THE ARMY, THE NAVY, AND THE AIR FORCE
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FOREWORD

The seismic design guidelines manual was developed to meet one of the objectives for earthquake hazards reduction measures as promulgated by the Earthquake Hazards Reduction Act of 1977 (Public Law 95–124). The objective is the development and implementation of a technologically and economically feasible, improved design and construction methods and practices in areas of seismic risk to provide earthquake resistant structures which are especially needed in time of disaster.

This guideline manual provides the latest seismic design concepts for earthquake resistant structures by utilizing the dynamic analysis approach. The concept is for essential buildings but also includes design provisions for high risk and irregular buildings. This manual also provides methodologies and procedures to determine site-dependent earthquake ground motions for sites anywhere in the United States. Two levels of earthquake motion are considered. At the first level, the structure will be designed to remain elastic for damage control at a moderate earthquake and at the second level, the criterion requires that the structure remains functional after a major earthquake. Also, commentary and design examples are included to provide a comprehensive applications of the design methodologies for earthquake resistant facilities.

The general direction and detailed development of this manual was under the supervision and guidance of the Office of the Chief of Engineers, Headquarters, Department of the Army, Washington, DC and necessary coordination was maintained with the Naval Facilities Engineering Command, Headquarters, Department of the Navy, Washington, DC and Directorate of Engineering and Services, Headquarters, Department of the Air Force, Washington, DC.

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CHAPTER 1
GENERAL

1-1. Purpose and scope.

a. Purpose. This manual prescribes criteria and furnishes guidelines for the design of essential buildings, high-risk buildings, and other structures that may require analytical procedures that are beyond the scope of TM 5-809-10/NAVFAC P-355/AFM 88-3, chapter 13, "Seismic Design for Buildings." Methodologies and procedures are given for determining site-dependent ground motion and for the dynamic analysis of buildings. These criteria apply to all elements responsible for design of military construction located in seismic regions. This manual is a supplement to TM 5-809-10/NAVFAC P-355/AFM 88-3, chapter 13, referred to herein as the Basic Design Manual.

b. Scope. Approval from DAEN-ECE-D (Army), NAVFAC Code 04BA (Navy), or HQ USAF/LEEE (Air Force) is required for the use of this manual as an alternative requirement to applicable provisions of the Basic Design Manual. This manual is for guidance in the design of buildings and other structures housing essential mission-oriented facilities and those that are vitally needed for post-disaster recovery that require continuous operation during and after an earthquake. This manual may also be used for guidance in the design of buildings that are classified in a high-risk category; buildings that are irregular in shape, size, and configuration that require consideration of the dynamic characteristics of the structure; and all other buildings as an alternative to the equivalent lateral static force procedure for determination and distribution of seismic forces. These guidelines encompass: (1) assessment of the seismic hazard at the site; and (2) seismic design of the structural and nonstructural systems for new buildings and other structures. The problems relating to earthquake-induced ground failure (e.g., liquefaction) are already stated in Basic Design Manual paragraph 2-7 and will not be covered in this manual. Alterations or evaluations of existing structures are not specifically covered by this manual; however, the principles and guidelines contained herein may be adapted for such use.

c. Seismic hazard risk levels. Seismic ground motion input for two risk levels is specified in chapter 3 for the prescribed structural performance criteria in chapter 4. The selected risk levels of the two earthquakes (EQ-I and EQ-II) are based on DOD standards; however, the risk levels may be revised, as warranted, by approval authorities.

d. Classification of structures.

(1) *Hazardous critical facilities.* These facilities (e.g., nuclear power plants, dams, and LNG facilities) are not included within the scope of this manual, but are covered by other publications or regulatory agencies. For any facilities housing hazardous items not covered by criteria, advice should be sought from DAEN-ECE-D (Army), NAVFAC Code 04BA (Navy), or HQ USAF/LEEE (Air Force).

(2) *Essential facilities.* These are structures housing facilities that are necessary for post-disaster recovery and require continuous operation during and after an earthquake. This includes facilities where damage from an earthquake may cause significant loss of strategic and general communications and disaster response capability. This category also includes facilities serving an essential military function that must not be disrupted. Typical examples are listed in the Basic Design Manual, paragraph 3-5*a*.

(3) *High-risk.* This classification includes those structures where primary occupancy is for assembly of a large number of people; where the primary use is for people that are confined; or where services are provided to a large area or large number of other buildings. Buildings in this classification may suffer limited damage in a large earthquake, but are recognized as warranting a higher level of safety than the average building. Typical examples are listed in the Basic Design Manual, paragraph 3-5*b*.

(4) *All others.* The provisions of this manual may be used for irregular buildings or as an option for all other buildings not covered by the above paragraphs only with the consent of the approval authority.

1-2. Background.

a. Expectations. Current seismic design criteria, such as prescribed by the Basic Design Manual, consist of specified equivalent lateral static forces that are resisted by the designed structural systems. Structures designed in conformance with such provisions and principles are expected to be able to: (1) resist minor earthquakes without damage; (2) resist moderate earthquakes without structural damage, but with some nonstructural damage; and

(3) resist major or severe earthquakes without major failure of the building or its component members and equipment, and to maintain life safety. For most structures, even in a major earthquake, structural damage should be limited to repairable damage. It is also recognized that for certain critical facilities, particularly those essential to public safety and well-being in case of emergency, criteria should be available to the designer that will permit design of a facility that will remain operational during and after an earthquake.

b. Lessons learned. Recent earthquakes have demonstrated that the existing seismic design requirements, as they have been implemented, are not necessarily adequate to insure continued operation of critical facilities vitally needed after a major earthquake, such as hospitals, fire stations, and communications centers. Therefore, there is a need for a more realistic approach to seismic-resistant design for buildings that must remain continuously functional after a major earthquake.

c. Recent developments. Earthquake engineering research and data collected from ground motion instrumentations and earthquake-caused building responses during the last two decades have greatly increased knowledge in geotechnical fields and have presented a clearer understanding of the performance of materials and structural elements. Therefore, practicing engineers are able to become familiar with methods of dynamic analysis as they are exposed to new design procedures by means of technical publications, conferences, and continuing education programs.

d. Design philosophy. One way of attempting to reduce the risk of earthquake damage to buildings is by imposing a higher design force coefficient, such as an I-factor of 1.5, for essential facilities. This is not always a sufficient or satisfactory approach to seismic design. Increasing the design forces by 50 percent may be insignificant if a major earthquake results in demands several times the design capacity. On the basis of current knowledge, it appears that a two-level (or two-phase) approach to design will give better insight to postulated behavior of structures. In this procedure, geotechnical data and probabilistic techniques are used to postulate the motion for two earthquakes: (1) the maximum probable earthquake, which is likely to occur one or more times during the life of the building (e.g., an earthquake with a 50-percent chance of being exceeded in 50 years); and (2) the maximum theoretical earthquake that can occur at the site, but has a low probability of occurring during the life of the building (e.g., 10-percent chance of being exceeded in 100 years). In the first phase of the procedure, the building is structurally designed to resist the lower level earthquake within prescribed bounds of elastic-linear procedures. In the second phase of the procedure, the building is analyzed for its response to the higher level earthquake by means of procedures that account for inelastic behavior, ductility demands, potential instability, and damage control. These guidelines are intended to insure that essential facilities will be capable of resisting the two levels of earthquake ground motion as follows: (1) for ground motion associated with the maximum probable earthquake, only minor damage, if any, will occur and the facilities will not have any loss of function; and (2) for ground motion associated with the maximum theoretical earthquake, no catastrophic failures will occur, damage will be repairable, and essential facilities will remain functional. The definitions and the methodology for determining these earthquakes are covered in chapter 3. The criteria and procedures for design are covered in chapters 4 and 5.

1-3. Preparation of project documents.

a. Design analysis. A design analysis conforming to agency standards will be provided with final plans. This design analysis will include seismic design computations for the determination of ground motion charateristics, for the determination of dynamic characteristics of the structure, for the stresses in the lateral-force-resisting elements and their connections, and for the resulting lateral deflections and interstory drifts. The first portion of the Design Analysis, called the Basis of Design, will contain the following specific information:

(1) A statement on the methodology used for determining the ground motion criteria, and a description of the response spectra for which the structure will be designed.

(2) A description of the structural system selected for resisting lateral forces and a discussion of the reasons for its selection. A symmetrically configured lateral resisting framing system, without vertical irregularities, will be required. However, if irregular conditions are unavoidable, a statement describing special analytical procedures to account for the irregularities will be submitted for review and approval by the approval authority.

(3) A statement regarding compliance with this manual, including a list of the values selected for damping and maximum inelastic demand ratios for critical structural elements.

(4) Any possible assumed future expansion for which provisions are made.

b. Drawings. Preparation of drawings will conform to agency standards for ordinary construction, with the following additional specific requirements for seismic construction:

(1) Preliminary drawings will contain a statement that seismic design will be incorporated in accordance with this manual. The Basis of Design will comply with paragraph *a* above.

(2) Construction drawings for seismic areas will include the following additional special information:

(a) A statement on the seismic ground motion criteria including the design peak ground accelerations and related response spectra.

(b) A statement on the lateral-force design criteria including a tabulation of the periods of vibration and equivalent design lateral forces and other factors.

(c) Assumptions made for future extensions or additions.

(3) Site adaption of standard drawings will include design revisions for the seismic area as required.

1-4. References and bibliography.

Publications that may be required to supplement the provisions of this manual are listed in appendix B, References. Publications that may be useful as back-up material and are presented as suggested reading are included in the bibliography. When pertinent to the subject, some publications in the bibliography are noted in the text by the bibliography number, in parenthesis.

CHAPTER 2
INTRODUCTION TO SEISMIC ANALYSIS

2-1. Introduction.

This chapter provides an introduction to the basic concepts of dynamic analysis for buildings responding to the ground motions caused by earthquakes. General guidance is given in the selection and use of various procedures for the design of structural systems.

2-2. General.

An earthquake causes vibratory ground motions at the base of a structure and the structure actively responds to these motions. Seismic design involves two distinct steps: (1) determining or estimating the forces that will act on the structure; and (2) designing the structure to resist these forces and to keep deflections within prescribed limits.

a. Determination of forces. There are two general approaches to determining seismic forces: (1) an equivalent static force procedure, such as presented in the Basic Design Manual; and (2) a dynamic analysis procedure. This manual illustrates the dynamic analysis procedure. Seismic forces are determined from data derived from the specification of ground motion. These ground motion data will generally be given in terms of a response spectrum; however, in some cases the data may be described in terms of a digitized time history.

b. Design of the structure. Structures are generally designed to resist applied forces well within the elastic capacity of their structural members. This is accomplished either by prescribing maximum allowable working stresses for materials, or by using a strength design concept with prescribed load factors. However, for exceptional loading conditions, such as caused by major earthquakes, structures may be required to resist deformations that exceed the elastic capacities of the structural elements. In conventional methods of seismic design, it is assumed that the design criteria will provide adequate safety by means of load factors and special details that provide the necessary ductility to resist major earthquake deformations. In the methods presented in this manual, the design procedures will give a better insight as to the performance of a structure when subjected to the exceptional loading conditions of a major earthquake. This method is generally referred to as a two-level approach to structural design.

2-3. Ground motion caused by earthquakes.

A general introduction to earthquake ground motion is presented in the Basic Design Manual. The relationship of a ground motion to the site and an introduction to time-history and response spectra are presented herein. A detailed methodology for determining site-specific ground motion characteristics is covered by chapter 3 of this manual.

a. General.

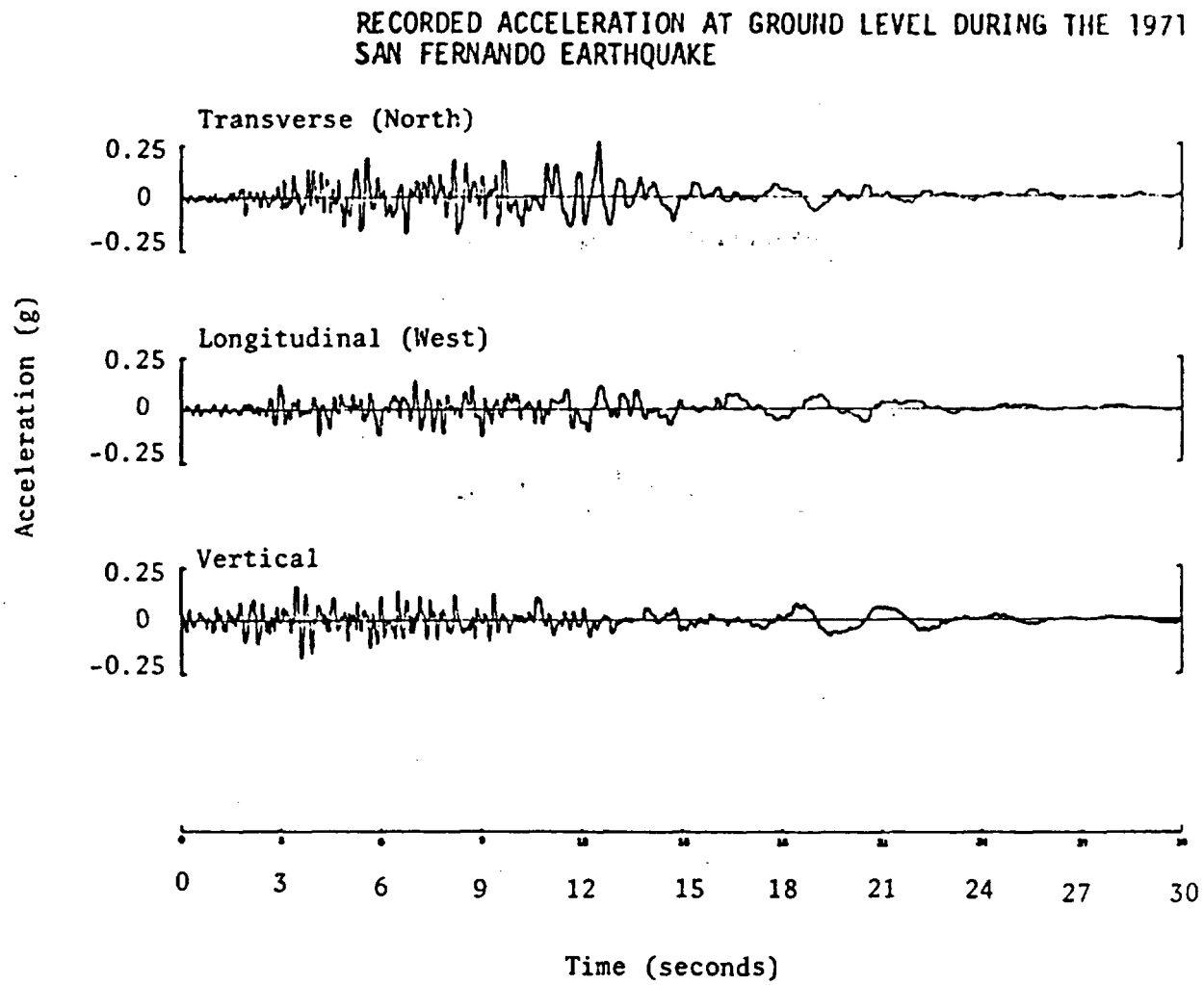
(1) Ground motion is generally strongest in the vicinity of its source (e.g., a rupturing fault), with the severity of shaking diminishing with an increase in distance.

(2) The predominant periods of ground motion vibration generally lengthen as distance increases from the source (para 3-6*f*).

(3) Deep deposits of soft soils tend to produce ground surface motions having predominantly long period characteristics.

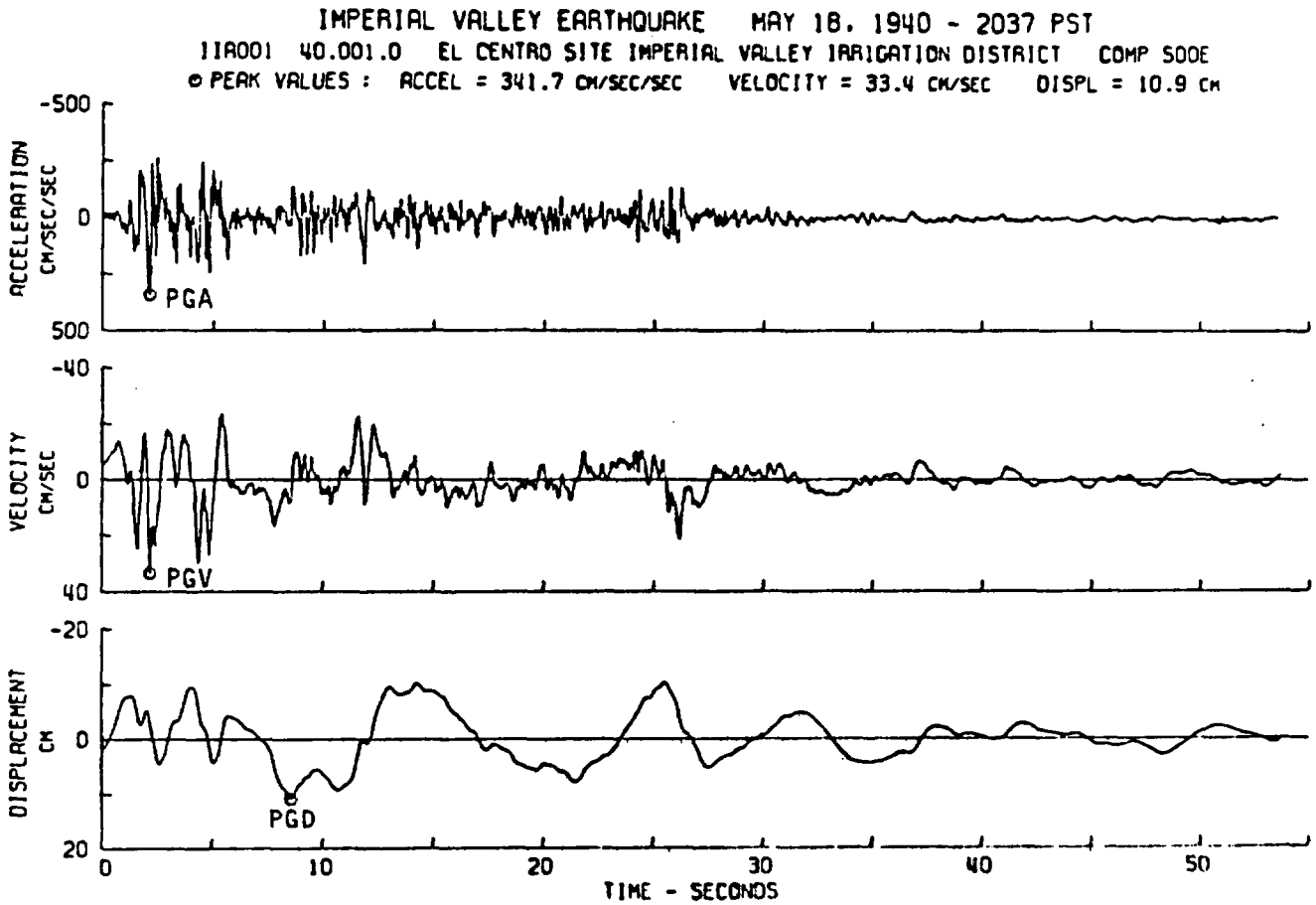
(4) Deposits of stiff soils or rock result in ground motions having predominantly short period characteristics.

b. Time history. The basic measurement of earthquake ground motion is the accelerogram record taken by seismometers. When these instrument records are properly corrected for elimination of recording noise and for base line adjustment, a primary data base for seismic load specifications is provided. Data banks of past earthquake records from all parts of the world are readily accessible from earthquake research centers. A typical seismometer station provides records of two orthogonal horizontal motions and one vertical motion, as illustrated in figure 2-1. The corresponding processed accelerograms are intended to be the best representation of the actual ground acceleration at the recording site. For a given component, the time derivative relations between ground displacement, $x(t)$; velocity $x(t)$; and acceleration, $\ddot{x}(t)$, allow the presentation of each of these motion histories, as shown in figure 2-2. The maximum or peak values of displacement (PGD), velocity (PGV), and acceleration (PGA) provide the most elementary and popular measures of an earthquake's severity. Duration (or bracketed duration) of strong motion is also an important measure, but it is not explicitly used in design criteria at the present time.



Reprinted from "The San Fernando, California, Earthquake of February 9, 1971," U.S. Government Printing Office, 1971.

Figure 2-1. Recorded acceleration at ground level for three components of motion.



Reprinted from "United States Earthquakes," U.S. Coast and Geodetic Surveys, U.S. Government Printing Office, 1940.

Figure 2-2. Ground acceleration and integrated ground velocity and displacement curves.

c. Response spectra. For design purposes, it would be ideal to forecast the acceleration time history of a future earthquake having a given hazard of occurrence. However, the complex random nature of an accelerogram makes it necessary to employ a more general characterization of ground motion. Specifically, the most practical representation is the earthquake response spectrum. This spectrum is used not only to describe the intensity and vibration frequency content of accelerograms, but also the most important advantage is that spectra from several records can be normalized, averaged, and then scaled according to seismicity to predict future ground motion at a given site. The physical definition of an acceleration response spectrum is shown in figure 2-3. A set of linear elastic single-degree-of-freedom (SDOF) systems having a common damping ratio, β, but each having different harmonic periods over the range O, T_1, T_2, etc. is subjected to a given ground motion accelerogram. The entire time history of acceleration response is found for each system, and the corresponding maximum value, S_a, is plotted

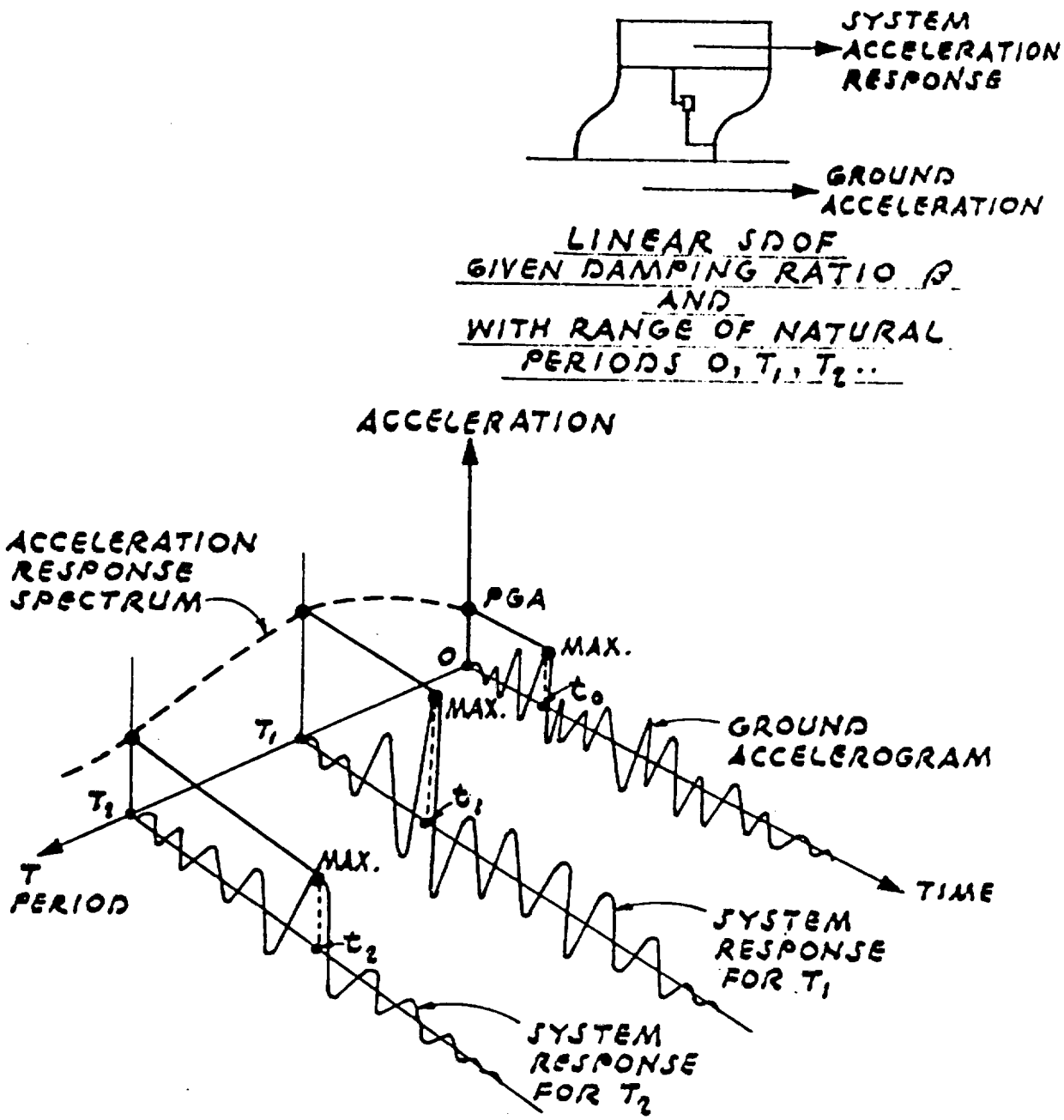


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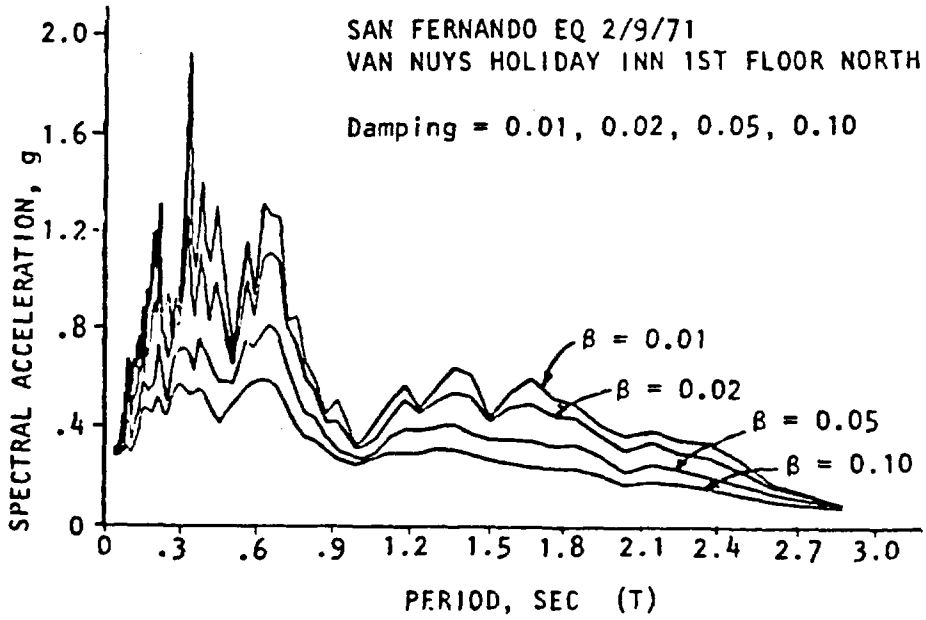
Figure 2-3. Description of acceleration response spectrum.

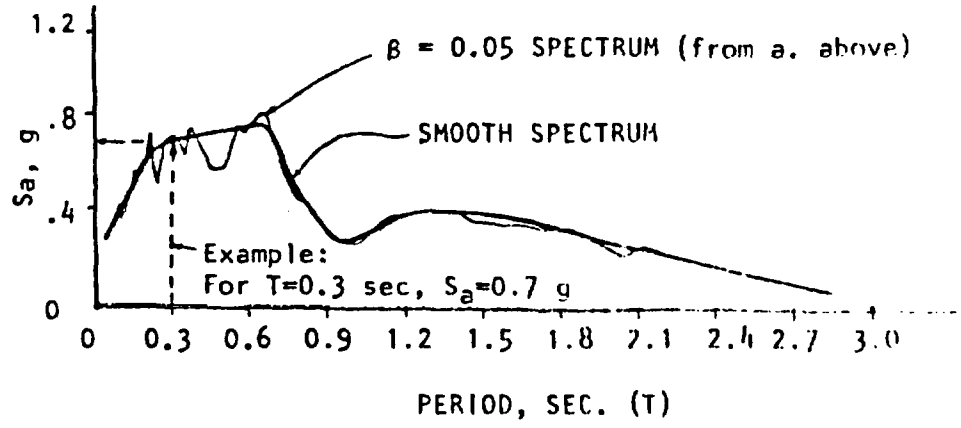
on the period axis for each system period. The curve connecting these S_a values is the acceleration response spectrum for the given accelerogram and damping ratio. Actual spectra for the transverse (north) accelerogram of figure 2-1 are shown for several damping ratios in figure 2-4. A smoothed individual spectrum (fig 2-4*b*), or averages of multiple record spectra, is employed as the seismic load input for the dynamic analysis of structures. Note that the S_a curve provides the maximum response value for any given system period, T.

2-4. Site effects.

a. Response spectrum shape. Response spectra shapes are determined largely by empirical data. Time history records of past earthquakes are used to construct response spectra. As the data bank increases, average trends can be observed with respect to the general shape of re-



a. RESPONSE SPECTRA: FOUR VALUES OF DAMPING (β)



b. SMOOTH RESPONSE SPECTRUM: β = 0.05

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Figure 2-4. Response spectra from recorded ground acceleration shown in figure 2-1.

sponse spectra curves. When these data are catalogued in terms of tectonic region, event intensity, distance, and site characteristics, specific response spectra shapes can then be developed that represent the conditions of particular sites. Procedures for developing response spectra are covered in chapter 3, and illustrative examples are included in appendix D.

b. Soil column. Site soil characteristics can be used to develop a mathematical model of a soil column at a building site. For a postulated bedrock earthquake, analytical procedures can be used to calculate the soil column's effect on the ground motion at the surface or the base of a structure. These results can be used either to calculate the shape of the response spectrum of these particular conditions, or used directly for time history analysis of the structure.

c. Foundation design. All inertia forces originating from the masses on the structure must be transmitted to and from the lateral-force-resisting elements, to the base of the structure, and into the ground. Foundations must be designed to provide stability for response due to maximum seismic ground motion. It should also be noted that the type, size, and depth of a foundation system can have an effect on a structure's response to seismic motion and that the actual seismic input is a series of reversing load cycles.

2-5. Dynamic analysis of structures.

Structures that are keyed into the ground and extend vertically some distance above the ground act either as simple or complex oscillators when subjected to earthquake-caused ground motion. Simple oscillators are represented by single-degree-of-freedom (SDOF) systems, and complex oscillators are represented by multi-degree-of-freedom (MDOF) systems. When a structure's base is suddenly moved by earthquake ground motion, the upper part of the structure will not respond instantaneously, but will lag behind because of the structure's inertial resistance and flexibility. This concept is illustrated in the Basic Design Manual, paragraph 2-4. As time progresses during an earthquake, the structure's various natural modes of vibration will be excited to peak amplitudes of motion as described by the response spectrum (para 2-3*c*).

a. Single-degree-of-freedom system. One fundamental system that is investigated by dynamic analysis is the simple oscillator or SDOF system, as shown in figure 2-5. Represented by a single lump of mass on the upper end of a vertically cantilevered pole or by a mass supported by two columns (part a of fig 2-5), this system is used in textbooks to illustrate principles of dynamics. It represents two kinds of real buildings: (1) a single-column structure with a relatively large mass at its top; and (2) a single-story frame structure with flexible columns and rigid roof system. In the idealized system, the mass (M) represents the weight (W) of the system divided by the acceleration of gravity (g) ($M = W/g$). The pole or columns represent the stiffness (K) of the system, which is a ratio equal to a horizontal force (F) applied to the mass divided by the displacement (δ) resulting from that force ($K = F/\delta$). If the mass is deflected and then quickly released, it will freely vibrate at a certain frequency, which is called its natural freuency of vibration. The period of vibration (T), which is the inverse of the frequency of vibration, is the time taken for the mass to move through one complete cycle (i.e., from one side to the other and back again (part b of fig 2-5). The period is equal to $2\pi\sqrt{M/K}$. In an ideal system having no damping ($\beta = 0$), the displaced system described above would vibrate forever. In a real system where there is some damping, the amplitude of motion will decrease for each cycle until the structure stops oscillating and comes to rest (part c of fig 2-5). The greater the damping, the sooner the structure comes to rest. The amount of damping is defined in terms of a ratio, or percentage, of critical damping. If the structure has damping equal to 100 percent of critical damping ($\beta = 1.0$), the displaced structure will come to rest without crossing the initial point of zero displacement. If oscillating motion is applied to the base of the system, the SDOF system will be forced to vibrate. If the oscillating motion at the base is at a period equal, or nearly equal, to the period of the SDOF system, the motion of the mass will amplify until it is substantially greater than the motion at the base. This condition is called resonance. The lower the value of β, the higher the amplification.

b. Multi-degree-of-freedom systems. Multistory buildings are analyzed as MDOF systems as shown in figure 2-6. They can be represented by lumped masses attached at intervals along the length of a vertically cantilevered pole (part a of fig 2-6). Each mass can be deflected in one direction or another; for example, all masses may simultaneously deflect in the same direction (the fundamental mode of vibration), or some masses may go to the left while others are going to the right (higher modes of vibration). An idealized system, such as shown in part a of figure 2-6, has a number of modes equal to the number of masses. Each mode has its own nat-

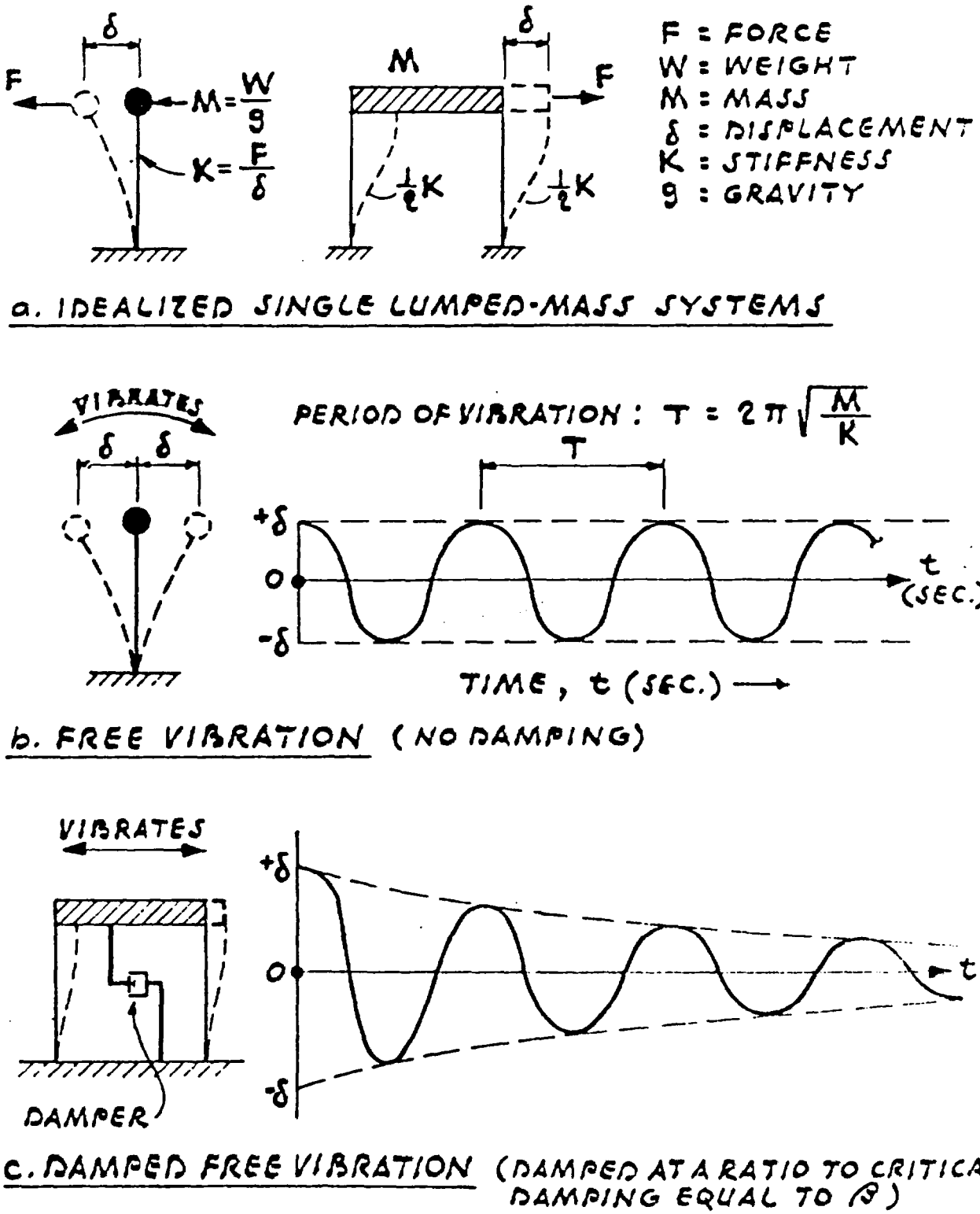

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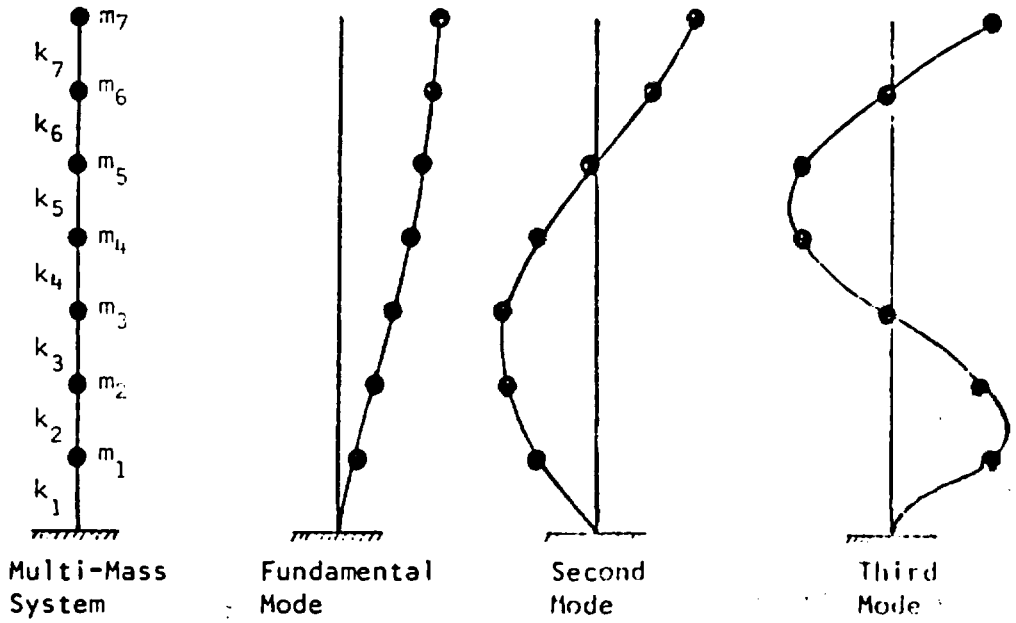
Figure 2–5. Single-degree-of-freedom system.



a. Idealized Lumped Mass System

b. Mode Shapes

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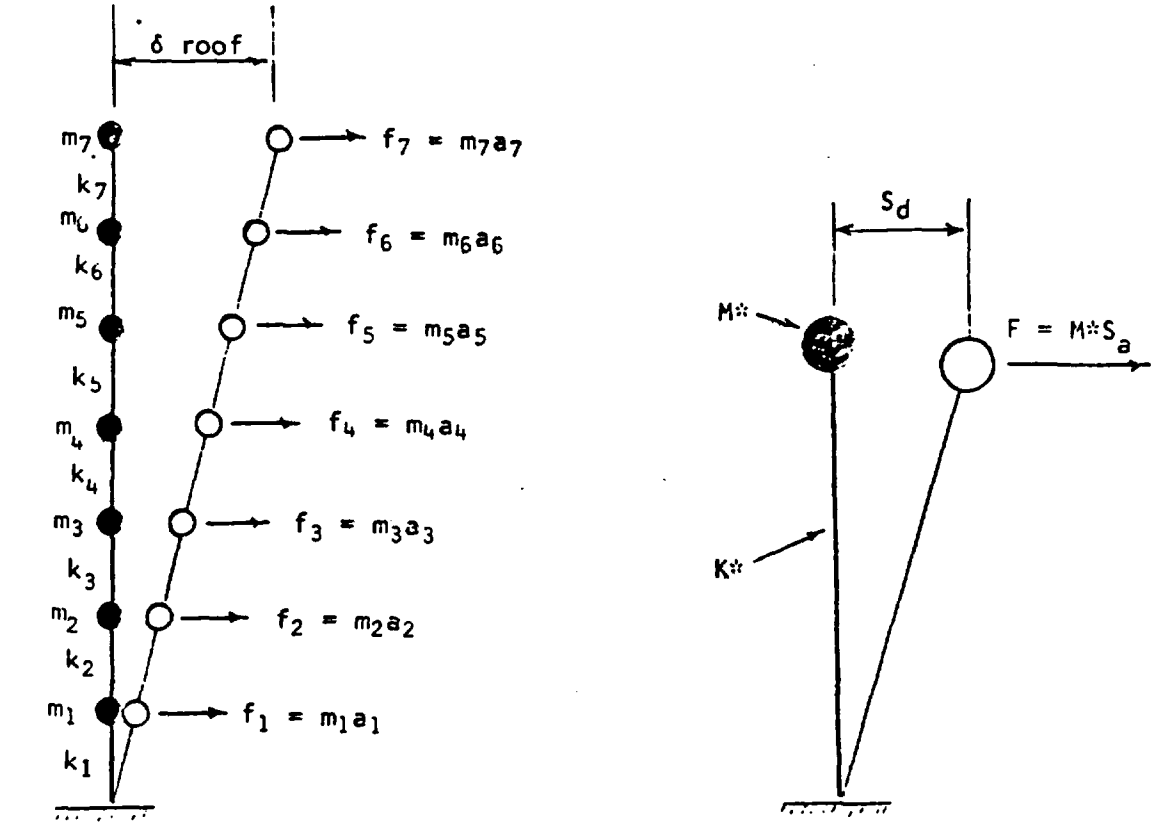
Figure 2–6. Multi-degree-of-freedom system.

ural modal period of vibration with a unique mode shape being formed by a line connecting the deflected masses (part b of fig 2–6). When oscillating motion is applied to the base of the multi-mass system, these masses move. The deflected shape is a combination of all the mode shapes; but modes having periods that are near, or equal to, predominant periods of the base motion will be amplified more than the other modes. Illustrative examples of MDOF systems are included in appendix E.

c. Multi-mode response to ground motion. Each mode of an MDOF system can be represented by an equivalent SDOF system having a normalized mass (M*) and stiffness (K*) where the period equals $2\pi\sqrt{M^*/K^*}$ (M* and K* are functions of mode shapes, mass, and stiffness). This concept, as shown in figure 2–7, provides the computational basis for using site specific earthquake response spectra based on SDOF systems for analyzing multi-storied buildings. With the period, mode shape, mass distribution, and response spectrum, one can compute the deflected shape, story accelerations, forces, and overturning moments. Using the response spectrum method on MDOF systems requires analyzing each predominant mode separately. Results of each individual modal analysis must then be combined in order to analyze the multi-mode system. For many buildings, the participation of the higher modes is negligible in relation to the participation of the fundamental modes of vibration. However, for tall, long-period, and irregular buildings, the second, third, and, possibly, higher modes may have a substantial effect. The amount of higher mode participation depends on both the building's modal characteristics and the amplitude-period characteristics of the response spectrum. Assuming that several modes are significant, one must select an appropriate method of combining the results of the several modes. One method is simply to add up the effects of each mode (absolute sum). This is an overly conservative approach because the response spectrum gives the peak response of each mode, and different modes reach their peak amplitudes at different times during the earthquake. Since the spectrum gives only the maximum values and the time of occurrence is unknown, some approximate method of mode combination must be used. The method most commonly employed is to combine the modes by the square-root-of-the-sum-of-the-squares (SRSS) of the peak response of each mode (this is analogous to a vector sum). This offers a reasonable value between the upper bound as the absolute sum of the modes and the lower bound as the maximum value of a single mode. To il-



First Mode of a Multi-Mass System

Equivalent Single-Mass System

M* and K* are normalized values of mass and stiffness that represent the equivalent combined effects of the story masses (M) and stiffnesses (K)

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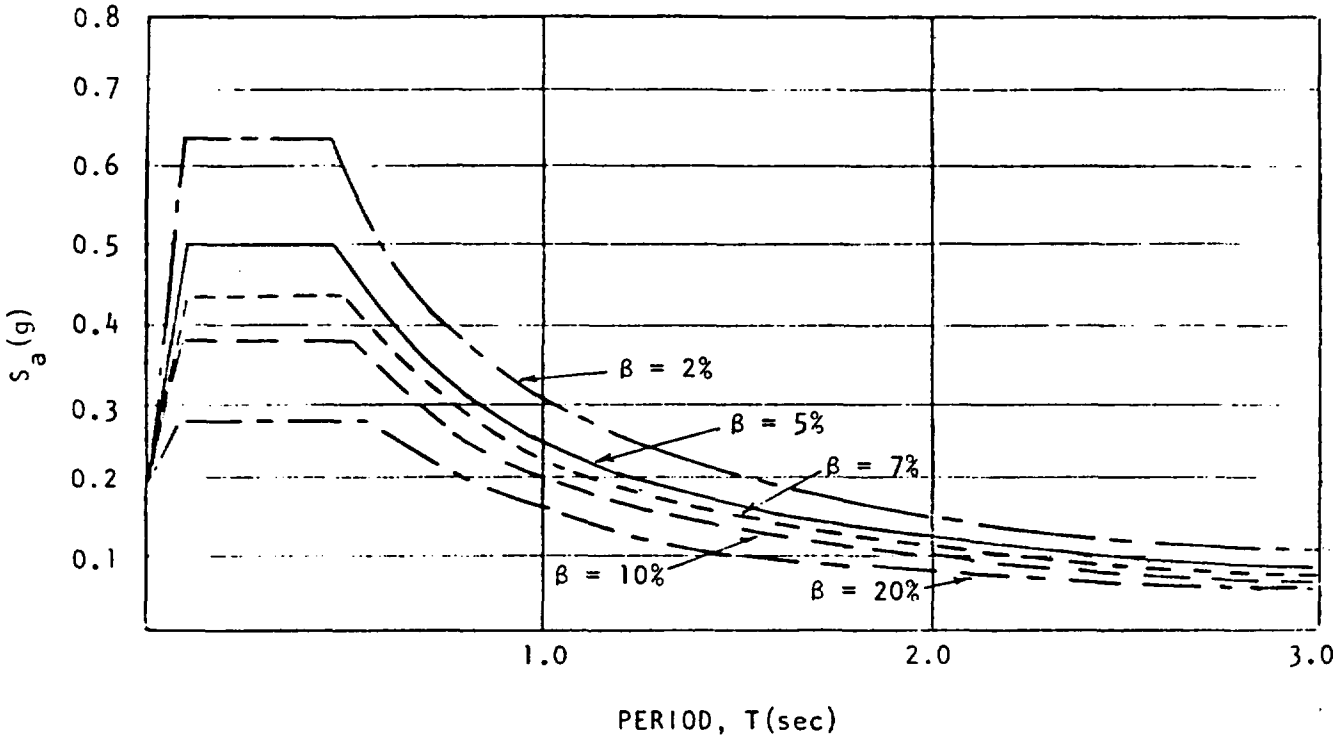
Figure 2-7. Multi-mass system represented by a single-mass system.

lustrate the multi-mode analysis of multi-storied buildings, two examples are given. Figure 2-8 shows design response spectra that are used for modal analysis examples of a 30-story building and a 7-story building.



SPECTRAL ACCELERATION, S_a(g)												
β \ T	0.1	0.48	0.50	0.80	1.0	1.25	1.5	1.75	2.0	2.25	2.5	3.0
2%	0.64	0.64	0.59	0.37	0.30	0.24	0.20	0.17	0.15	0.13	0.12	0.10
5%	0.50	0.50	0.48	0.30	0.24	0.192	0.16	0.137	0.12	0.107	0.096	0.08
7%	0.44	0.44	0.44	0.28	0.22	0.18	0.15	0.13	0.11	0.10	0.09	0.07
10%	0.38	0.38	0.38	0.25	0.20	0.16	0.13	0.11	0.10	0.090	0.08	0.066
20%	0.27	0.27	0.27	0.20	0.16	0.12	0.10	0.09	0.08	0.07	0.06	0.05

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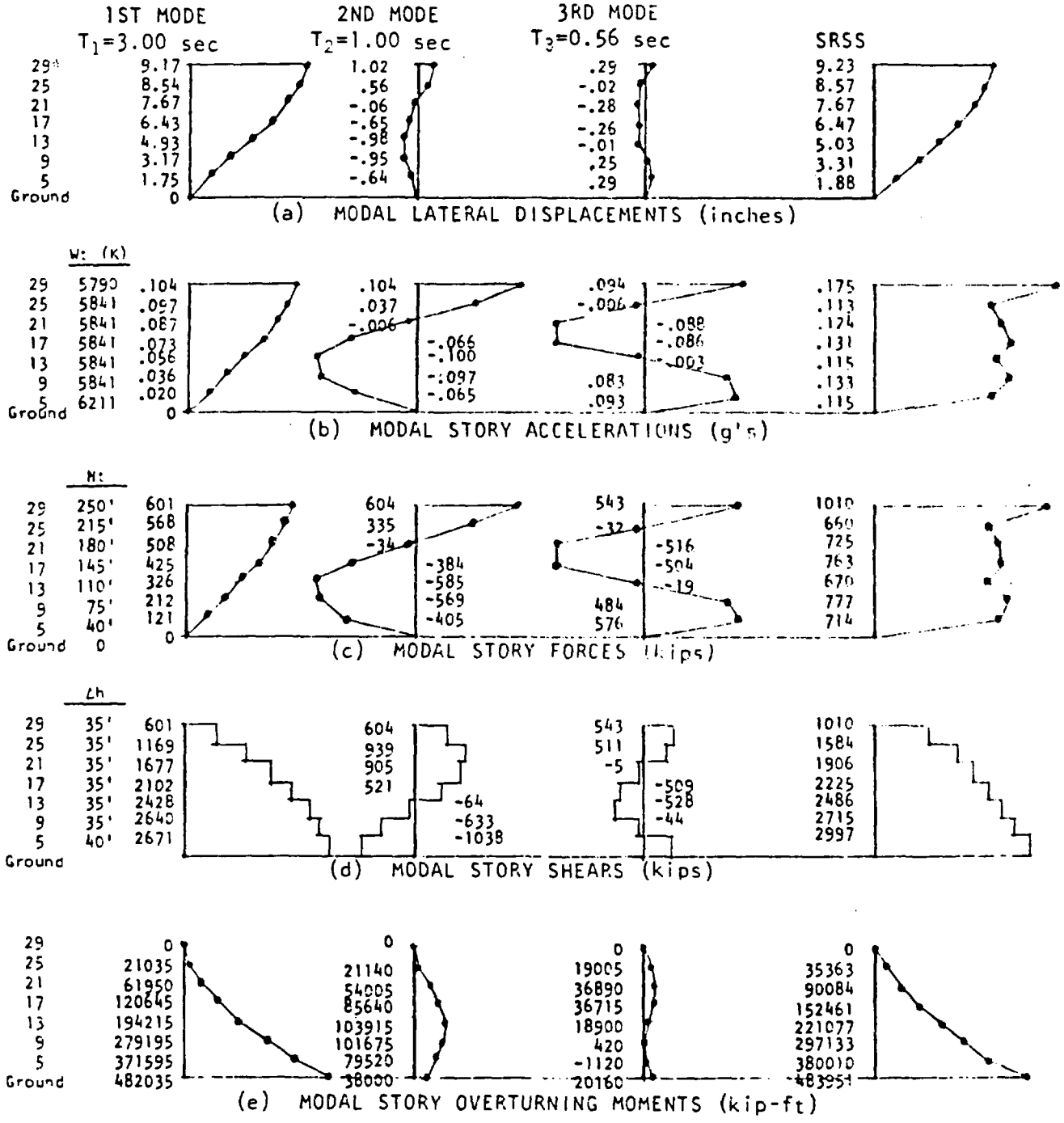
Figure 2-8. Design response spectra for examples in figures 2-9 and 2-10.

(1) *Thirty-story building.* The example in figure 2-9 summarizes the results of a modal analysis of a structural framing system that represents one principal axis of a 30-story building. The fundamental period of vibration is 3.0 seconds. The periods of the second and third modes of vibration are 1.00 seconds and 0.56 seconds, respectively. From the response spectrum curve in figure 2-8, which represents 5 percent of critical damping (β = 0.05), it is determined



*Story 29 represents the roof, floors 29 and 28, and one-half of floor 27. Other story designations represent the reference story plus one-and-one-half stories above and below.

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Figure 2-9. Sample modal analysis of a 30-story building.

that the second mode spectral acceleration (0.240g) is triple that of the first mode spectral acceleration (0.080g), and that the third mode spectral acceleration (0.45g) is over 5 times that of the first mode spectral acceleration. On the basis of mode shapes and modal participation factors (chap 5), modal story displacements, accelerations, forces, shears, and overturning moments can be determined. For ease of comparison to the 7-story example (para (2) below), the 30-story building is compacted to seven lumped masses, each representing four stories. Back-up data for this example are included in appendix E (design example E-1). The modal analysis procedure is covered in chapter 5.

(a) Diagram (a) of figure 2-9 shows the modal displacements. Note that the fundamental mode (first mode) predominates, while second and third mode displacements are relatively insignificant. The SRSS combination does not differ greatly from the fundamental mode.

(b) Diagram (b) shows story accelerations. In this form, the second and third modes do play a significant role in the structure's maximum response. While the shape of an individual mode is the same for displacements and accelerations, accelerations are proportional to displacements divided by the squared value of the modal period, which accounts for the greater accelerations from the higher modes. The shape of the SRSS combination of the accelerations is substantially different from shapes of any of the individual modes because it accounts for the predominance of the various modes at different story levels. Note that the maximum accelerations on stories 5 through 25 do not vary by more than 10 percent from the mean value, indicating that the maximum acceleration felt at most floor levels is fairly constant. However, these maximum values would not occur simultaneously or with the same period content.

(c) Diagram (c) shows story forces whose values are obtained by multiplying the story acceleration by the story mass (or weight). The shapes of diagram (c) curves are quite similar to the shapes of diagram (b) curves because the building mass is essentially uniform.

(d) Diagram (d) shows story shears, which are a summation of the modal story forces in diagram (c). The higher modes become less significant in relation to the first mode because the forces tend to cancel each other due to the reversal of direction. Except for the top stories, the SRSS values do not differ substantially from the first mode values.

(e) Diagram (e) of figure 2-9 shows the building overturning moments. Again, the higher modes become somewhat insignificant because of the reversal of force directions. The SRSS curve is essentially equal to the first mode curve at the lower stories of the building.

(2) *Seven-story building.* The example in figure 2-10 summarizes the results of a modal analysis of a structural framing system that represents one principal axis of a 7-story building. Back-up data for this example are included in appendix E (design example E-1). The periods of vibration are roughly 30 percent of the periods of the 30-story building (fig 2-9); periods of the first, second, and third modes being 0.880 seconds, 0.288 seconds, and 0.164 seconds, respectively. From the 5-percent damped response spectrum (β = 0.05) of figure 2-8, both the second and third mode spectral accelerations (0.500g) are 80 percent greater than the first mode spectral acceleration (0.276g).

(3) *Comparisons.* By comparing figures 2-9 and 2-10, it can be seen that the influences of the second and third modes in relation to the first mode are larger for the 30-story building than for the 7-story building. For taller buildings with longer periods of vibration, the influences of the higher modes may become larger, and participation of additional modes of vibration (e.g., fourth and fifth modes) may become significant.

d. Response of irregular buildings. When buildings are eccentric or have areas of discontinuity or other irregularities, the behavioral characteristic are very complex; whereas buildings with symmetrical shape, stiffness, and mass distribution and with vertical continuity and uniformity behave in a fairly predictable manner. In addition to the single axis of response shown in figures 2-9 and 2-10, the torsional response (twisting about a vertical axis) as well as the interaction or coupling of the two translational directions (longitudinal and transverse axis) of response must be considered. For example, the predominant motion may be skewed from the apparent principal axis. This is somewhat analogous to a Mohr's circle for principal stresses. Thus, three-dimensional methods of analysis are required and each mode shape is defined in three dimensions by the longitudinal movement, the transverse movement, and the angle of rotation. In addition to complicating the method of analysis, building irregularities complicate the methods used to combine modes. Methods such as SRSS may not be appropriate for some three-dimensional methods of dynamic analysis. Procedures for performing three-dimensional analyses are covered in chapter 5.

e. Inelastic-nonlinear response. In order to

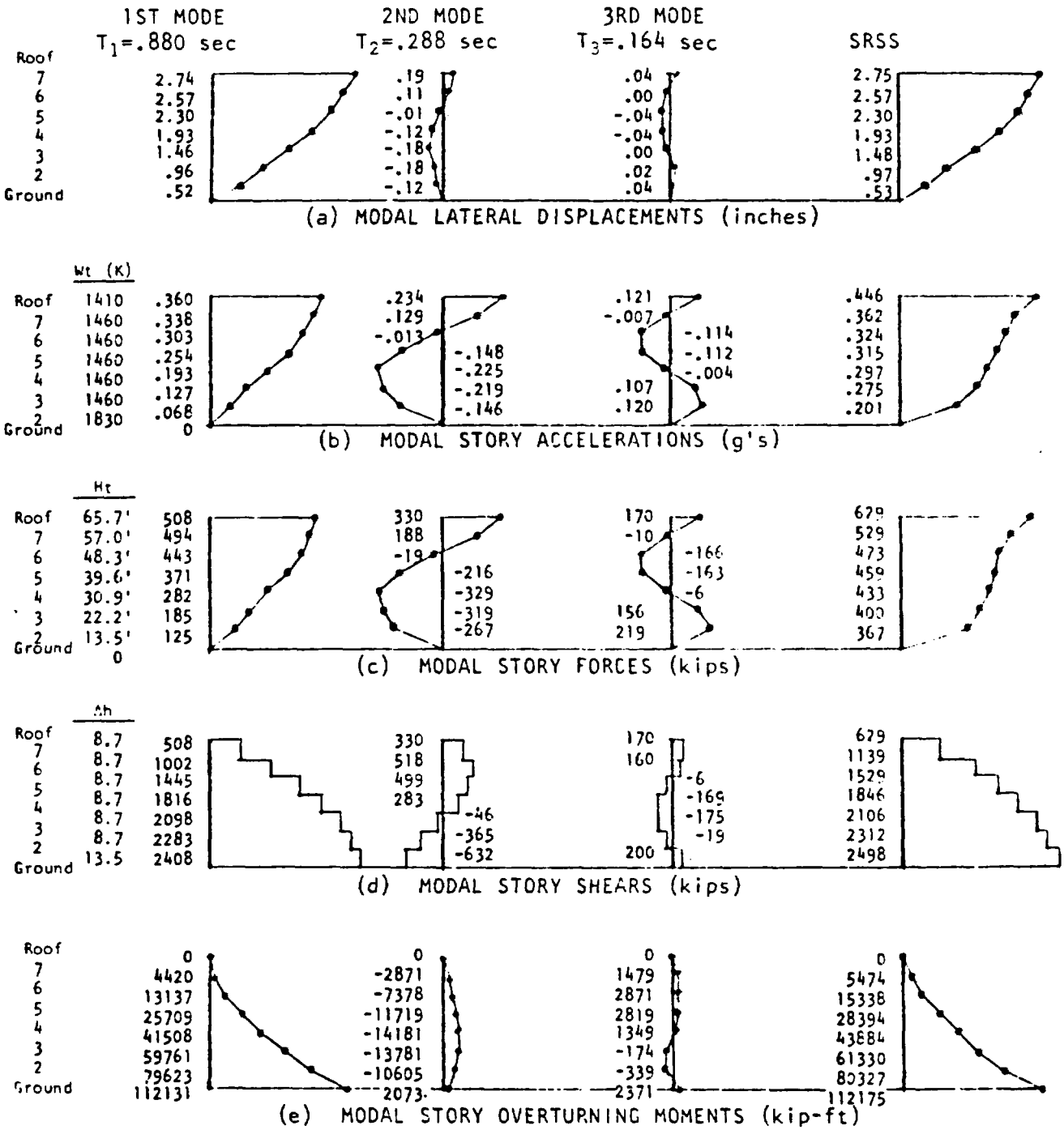

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Figure 2-10. Sample modal analysis of a 7-story building.

estimate the behavior of a structure that may be subjected to a major, damaging-type earthquake, it is necessary to investigate its inelastic response characteristics and capacity. The general procedures discussed in paragraphs a through d above are on the basis of elastic-linear distortions of the building's structural elements. When one major structural element begins to yield, changes will begin to occur in the structure's behavioral characteristics. For example, force distribution, periods of vibration, and mode shapes will be altered as parts of various elements yield. Dynamic analysis procedures for nonlinear systems can be very complex, requiring step-by-step, time-history-forcing-functions, and inelastic force-distortion properties of all the structural elements and their connections. However, approximate methods have been developed that give rough approximations as to the inelastic response or capacity of structures. Post-yield analysis procedures are discussed in chapter 5 and illustrative examples are included in appendix E.

2–6. Nonstructural elements.

Elements that are housed in the building, as well as portions of the building that are not part of the structural system, must also be investigated for their response to earthquake motion. These elements are generally categorized as architectural, mechanical, or electrical (refer to Basic Design Manual, chaps 9 and 10).

a. Elements attached to floors of buildings. These elements (e.g., mechanical equipment, free-standing partitions, storage racks, suspended fixtures) respond to floor motion in much the same manner as a building responds to ground motion. However, the floor motion may vary substantially from the ground motion. The high-frequency components that make the ground motion complex tend to be filtered out at the higher floor levels, while the components of motion corresponding to the building's natural periods of vibration tend to be magnified. In other words, a response spectrum of a building's floor motion will have predominant peaks at the participating periods of the building. If elements are rigid and rigidly attached to the structure, the maximum accelerations will be the same as the maximum floor accelerations, such as those shown in the SRSS curve of diagram (b) in figures 2–9 and 2–10. But, if the elements are flexible and have periods of vibration close to any of the predominant building vibration modes, these elements will experience accelerations substantially greater than the floor accelerations. Generally, a time–history analysis is required to determine the peak response of flexible or flexibly attached equipment at upper levels of a building. A time-history of the ground motion is used to calculate a time-history of the floor motion. The floor motion time-history is then used to construct a floor response spectra. This procedure is illustrated in figure 2–11. In chapter 6, an approximate method is shown for constructing design floor response spectra. Illustrative examples are included in appendix F.

b. Elements attached to adjacent floors. Elements extending vertically from floor to floor (e.g., full-height partitions, exterior panels, piping) will be subjected to two types of dynamic motion. One type is the response motion described in paragraph *a* above. The other type is due to the distortion resulting from the interstory displacements between two adjacent story levels. Interstory displacements for each mode can be obtained by finding the difference between adjacent modal lateral story displacements (diagram (a) in figs 2–9 and 2–10). Interstory displacements for a multi-mode system can be approximated by combining the modal interstory displacements by the SRSS or other methods.

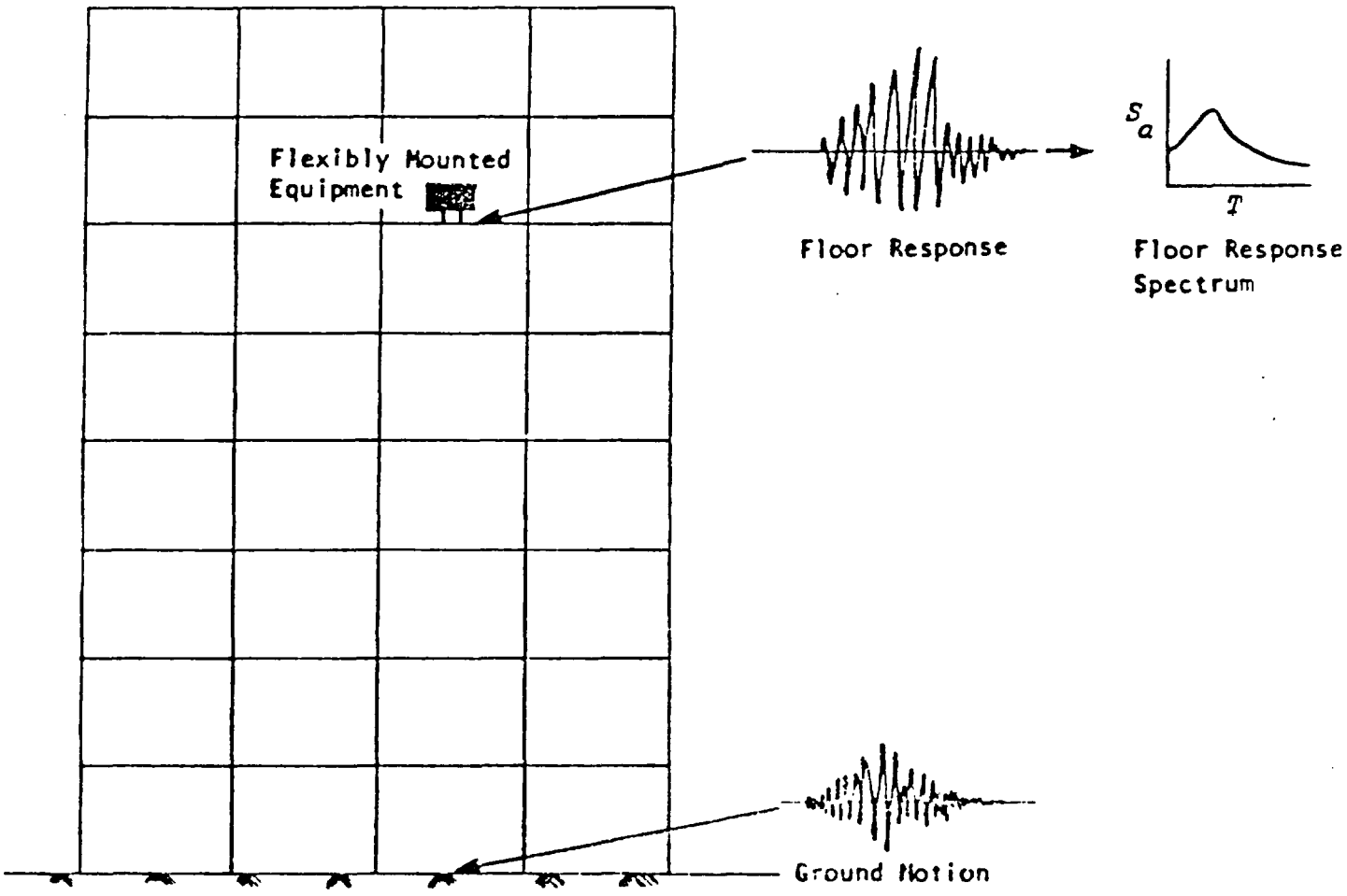

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Figure 2-11. Response of flexibly-mounted equipment in buildings.

CHAPTER 3
SPECIFICATION OF GROUND MOTION

Section I BASIC STEPS FOR SPECIFICATION OF GROUND MOTION

3-1. Introduction.

The purpose of this chapter is to describe the methodologies for determining site dependent earthquake ground motions for sites anywhere in the United States. The objective is to develop design parameters from the available information and seismic ground motion. The principal method of describing these ground motions will be in the form of acceleration response spectra for input in the dynamic analysis of a given structure.

a. Selected method of description. There are several methods of arriving at a description of future earthquake loading. These are described briefly along with their advantages and disadvantages in appendix C, paragraph C-3. The method employing an attenuated site severity factor (such as peak ground acceleration, PGA) which is used to scale a normalized site spectral shape (Dynamic Amplification Factor, DAF) is judged to be the most appropriate and practical input for the dynamic analysis of building structures and therefore will be the principal method for this manual. However, this empirical method may be supplemented by available results from the other methods; particularly any findings from a site soil column response study, as described in appendix C, paragraph C-3.

b. Procedures. The following selection procedures will be followed for the evaluation of site dependent earthquake ground motions, (see fig 3-1). These procedures are dependent upon three conditions: the geotectonic regions of the Western United States (WUS) and the Eastern United States (EUS) as defined in paragraph 3-4*a*, the proximity of seismic sources, and the site soil conditions as described in table 3-5.

(1) For sites located within 20 kilometers from a fault or area source in the WUS, or within a tectonic province in the EUS, where the source or province has a maximum local magnitude of 6.0 or greater, the detailed procedures of paragraphs 3-3 through 3-7 will be considered and employed as directed by the responsible agency.

(2) For sites in either the WUS or EUS having normal site soil conditions conforming to the description of soil profile types S_1 or S_2 as described in table 3-5 and having locations outside of the limits of paragraph 3-1*b*(1), the ATC 3-06 method of section III, paragraph 3-8 of this manual may be used.

(3) For sites in the WUS having exceptional soil conditions conforming to the soil description of soil profile S_3 as described in table 3-5, the selection of the corresponding site specific response spectrum shape will consider and employ the recommendations of paragraphs 3-6*c*(3) or 3-6*f*(3) as directed by the responsible agency. If this WUS site location is outside of the limits of paragraph 3-1*b*(1), then the selected spectrum shape may be scaled by the appropriate site acceleration coefficient A_v given in paragraph 3-8.

(4) For sites in the EUS having the soil conditions conforming to soil profile S_3 and outside of the limits of paragraph 3-1*b*(1), the method of paragraph 3-8 may be used.

(5) In all cases where methods other than those of paragraph 3-8 are employed, the results will be compared with those from paragraph 3-8, and any significant differences will be justified and resolved. All final recommendations shall be subject to approval by the responsible agency.

c. Scope. The scope of this part of the Manual includes the description of the essential steps and related procedures necessary for the specification of site specific ground motion. These are listed in paragraph 3-3 for the Western United States (WUS) and the Eastern United States (EUS), and for the deterministic and probabilistic procedures.

d. Current state-of-the-art. It is important to recognize that the field of ground motion specification is in a state of evolution. The general steps and input variables as outlined in this manual are reasonably well accepted by most of the researchers and users. However, because of the very active state of development, it is not possible to outline a step by step procedure which will remain the same with time as well as from region to region. Thus, the steps outlined in this manual are to be viewed as guidelines rather than as one universally accepted and recommended procedure.

e. Format of results. Various methods for the evaluation of the level of ground motion and its time history or frequency content are de-

Nothing in this chapter will prevent substantiated alternative methods or time history procedures if approved by the agency command.

Western United States (WUS)

Source to Site Surface Distance

Soil Type	I 20 Kilometers or less*	II More than 20 Kilometers
S_1 or S_2	Site Specific Hazard Analysis (para 3-3 to 3-7)	ATC 3-06 Method (para 3-8)
S_3	Same as above	Site Specific Spectra Development (para 3-6). Site Specific Hazard Analysis not Required.

* If line fault or area source, then source must have maximum M_{max} greater than 6.0, otherwise use Column II.

Eastern United States (EUS)

Soil Type	I Within a province having $M_{max} \geq 6.0$	II All regions other than in Column I
S_1 or S_2	Site Specific Hazard Analysis (para 3-3 to 3-7)	ATC 3-06 Method (para 3-8)
S_3	Same as above	Same as above

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Figure 3-1. Selection procedure.

scribed in appendix C, paragraph C-3. Of all these methods, the empirical method consisting of an PGA scaling factor for ground motion severity at a given risk level, and an effective DAF spectral shape, has been selected for the typical conditions and design objectives of this manual. An effective response spectrum will be specified for each of the two levels of structural performance. Unless specified by the appropriate agency the acceptable risk of exceedance will correspond to:

(1) A fifty percent risk of exceedance in fifty years (EQ-I), and

(2) A ten percent risk of exceedance in one hundred years, (EQ-II).

Table 3-1 shows the relationship between the exposure time (or economic life of the facility), the probability of exceedance and the return period.

3-2. Definition of Terms, Glossary, and Symbols.

The methodologies of determining ground motion are based on the following disciplines: geology, seismology, dynamics and vibrations, probability and statistics. Because of this rather extensive range of subject matter, it is necessary to provide both symbols and a glossary of terms used in this manual along with the related terminology commonly used in the references and necessary bibliography. These are given in appendix A, Symbols and Notations; and in the Glossary.

3-3. General Overview of Seismic Hazard Analysis and Specification of Ground Motion.

For engineering design and planning purposes, the future earthquake loadings at a site of interest must be known. The procedures and steps

Table 3-1. Return period as a function of exposure time and probability of non-exceedance

Exposure Time Years / "Hazard" or Probability of exceeding %	10	20	30	40	50	100
5	195	390	585	780	975	1950
10	95	190	285	390	475	950
20	45	90	135	180	225	449
30	29	57	84	113	140	281
40	20	40	59	79	98	196
50	15	29	44	58	72	145
60	11	22	33	44	55	110
70	9	17	25	34	42	84
80	7	13	19	25	31	63
90	5	9	14	18	22	44
95	4	7	11	14	18	34
99	3	5	7	9	11	22
99.5	2	4	6	8	10	19

for estimating this future loading comes under the general category of seismic hazard analysis. It should be recognized that there are two different approaches: deterministic and probabilistic. Deterministic approaches do not take into account the uncertainty in the size, the location, and the frequency of seismic events. Probabilistic approaches incorporate uncertainty in all the above quantities. An overview of the procedures for deterministic and probabilistic approaches is given in this paragraph. Steps are outlined by means of flow diagrams and illustrative formats. These are shown in figure 3-2 for the two main tectonic regions; the Western United States (WUS) and the Eastern United States (EUS).

a. *Algorithm of Basic Steps of Seismic Hazard Analysis.* Various earthquake severity parameters at the source and site are described in appendix C, paragraph C-1. The particular parameters (such as magnitude, intensity, and spectra) to be employed are dependent upon the type of information available to the analyst and the needs of the designer. The procedures and the models selected depend on the type, quantity, and quality of information as well as the goal of the analysis. The general procedures for evaluating seismic ground motion in the Western United States do not differ greatly from those in the Eastern United States. However, since the tectonic setting and the available seismic information varies greatly between those two geographic regions, the elements of the procedures are different. A discussion related to selection of deterministic or probabilistic procedures will be given in paragraph 3-3c. The five basic steps required for the evaluation of a site specific seismic ground motion are described below (see fig 3-3). The region-specific flow dia-

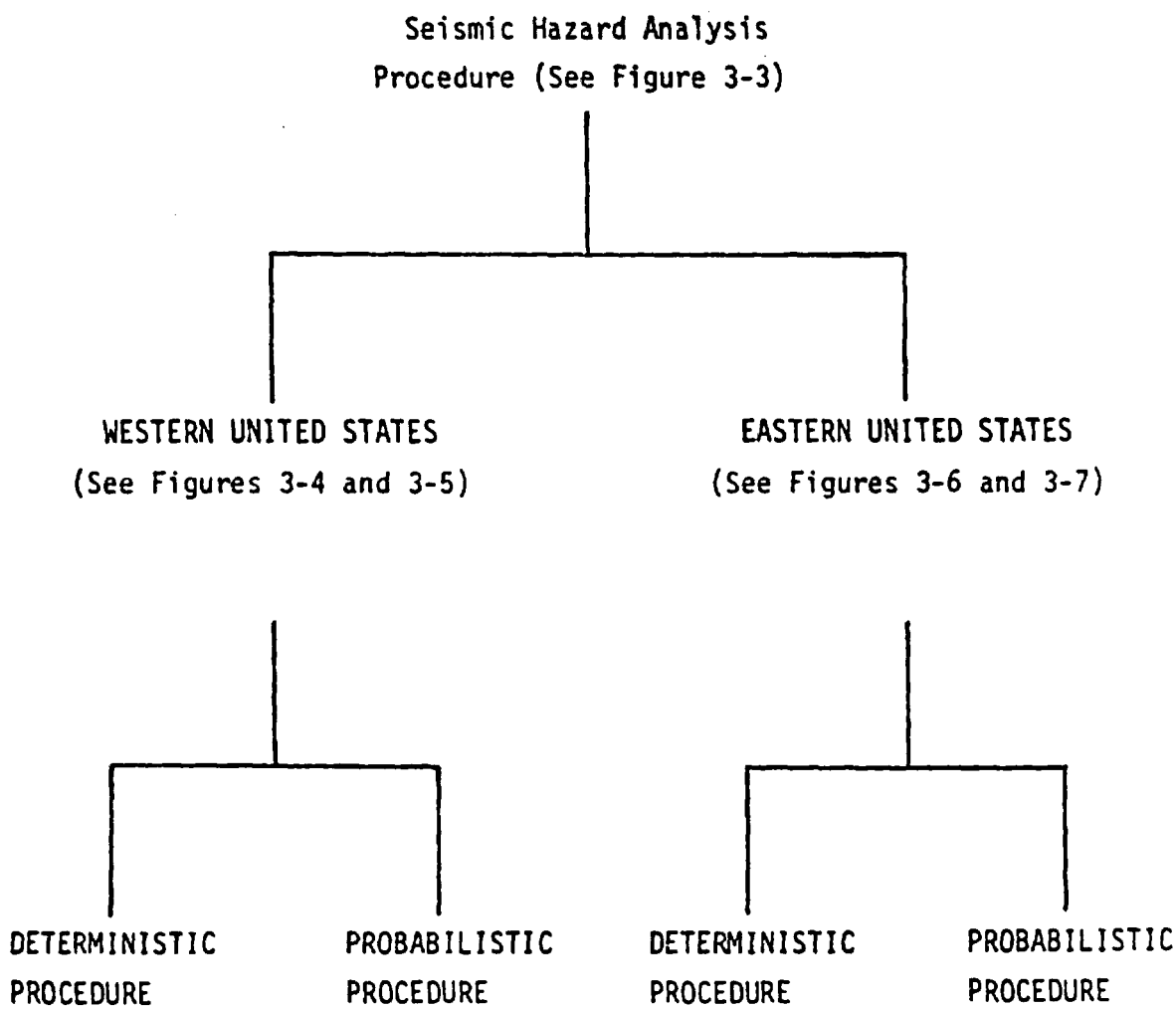

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Figure 3-2. General flow diagram selection chart.

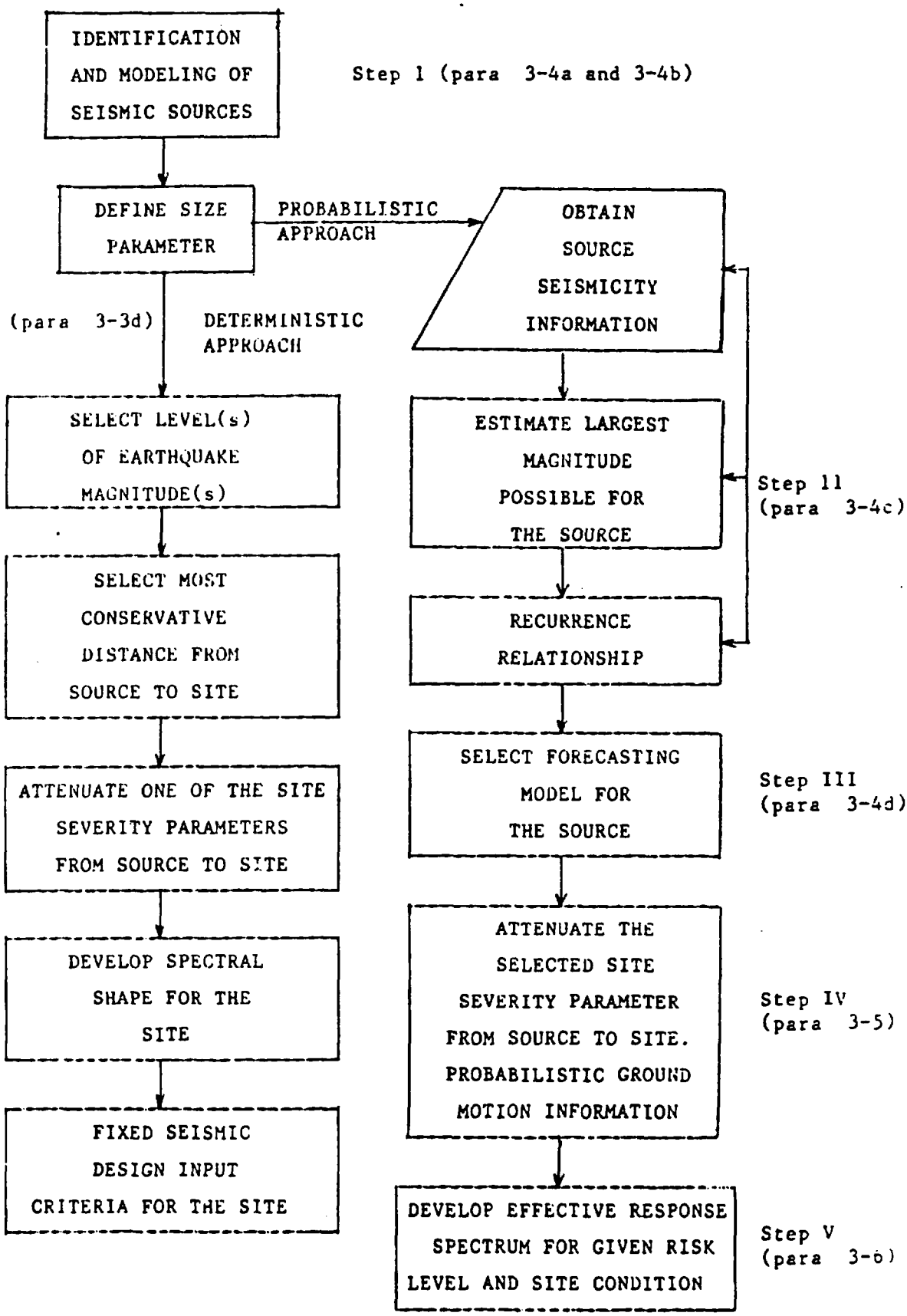

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Figure 3-3. General flow chart.

grams and illustrations of related procedures are shown in figures 3-4 and 3-5 for the WUS and figures 3-6 and 3-7 for the EUS. Each figure shows the parallel basic steps as required in the deterministic and probabilistic procedures.

(1) Step I is to identify and model seismic sources. The selected type and accuracy of this modeling depends on the available geologic, geotectonic, geomorphic, historic, and subjective information from experts. The purpose of this step is to assemble the information required to delineate faults and regions within which seismic activity can be considered homogeneous. See paragraph 3-4*b* for a detailed discussion and appendix D for examples.

(2) Step II is to define the size or severity parameter of the seismic event at the source and the related recurrence relation. The size will be

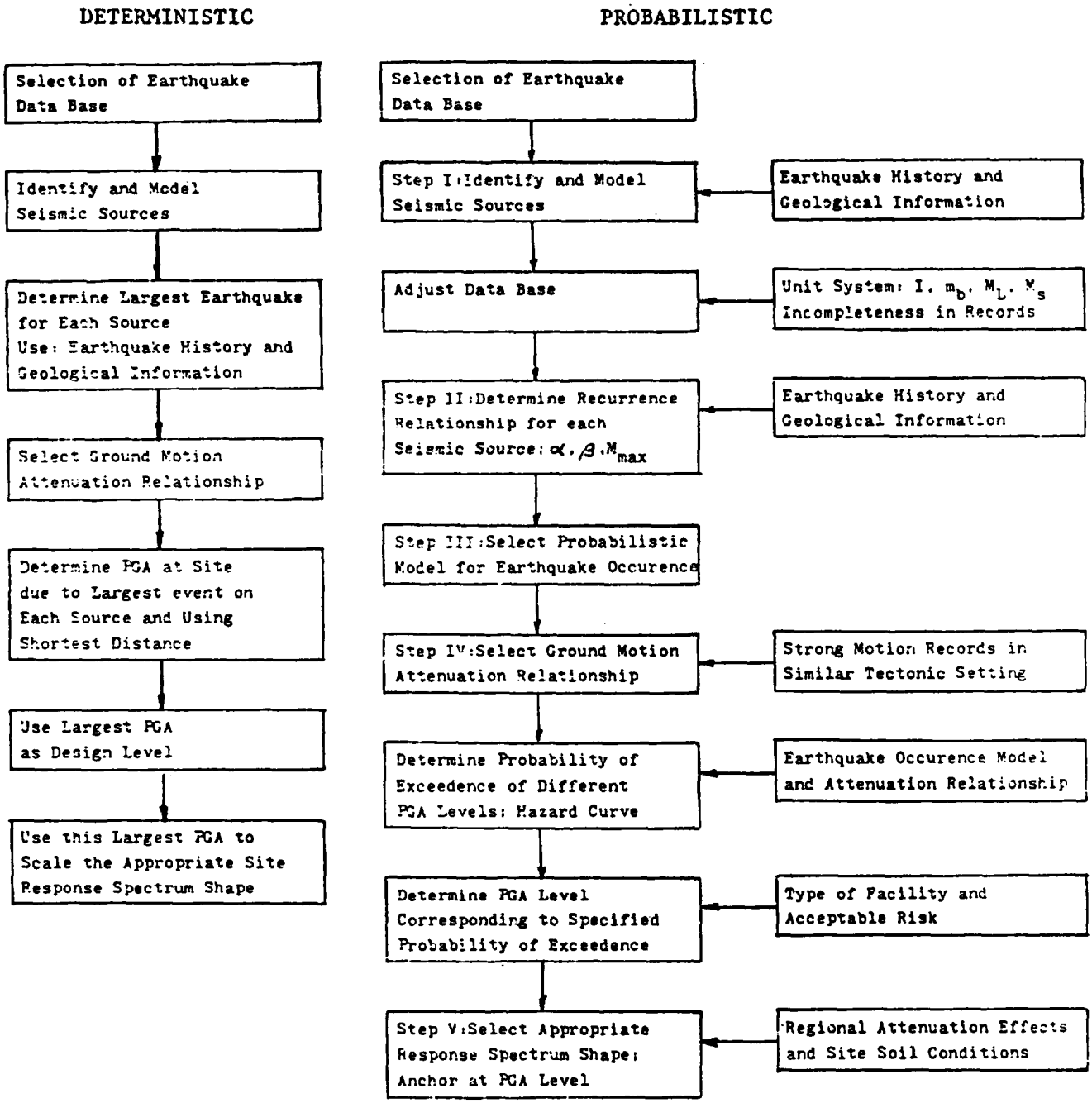

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Figure 3-4. Flow diagram for the Western United States.

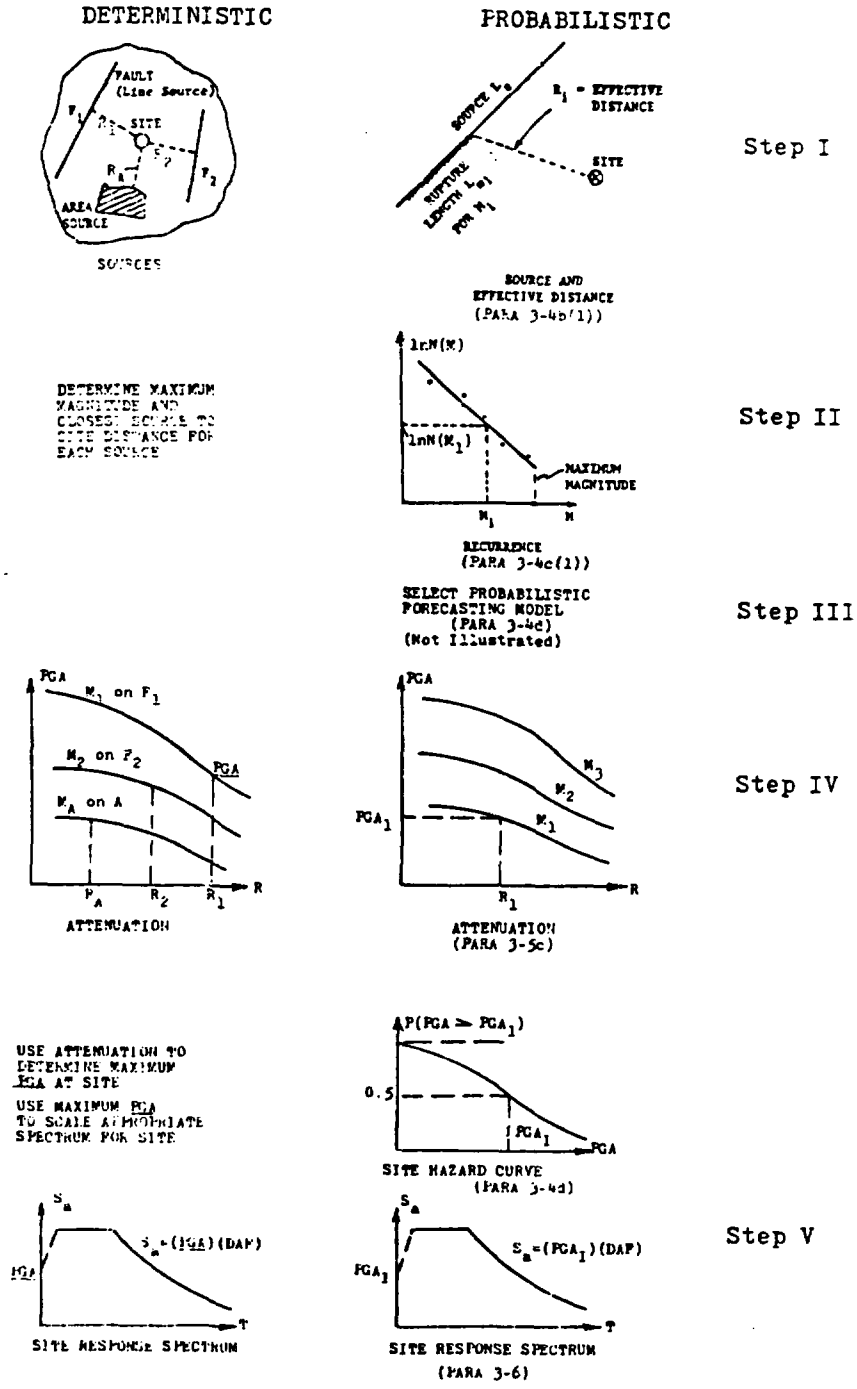

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Figure 3-5. Hazard evaluation of WUS.

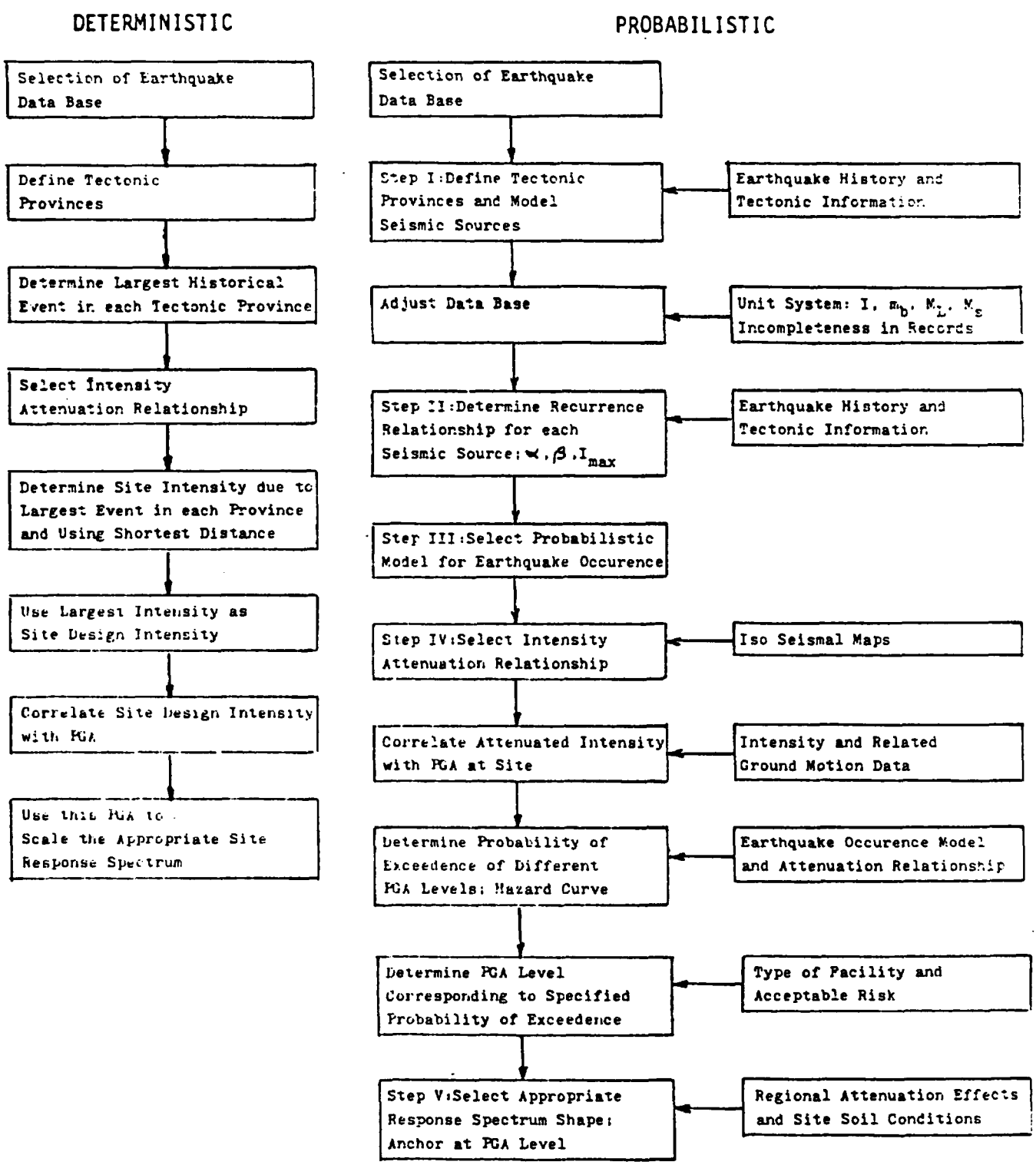

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Figure 3-6. Flow diagram for the Eastern United States.

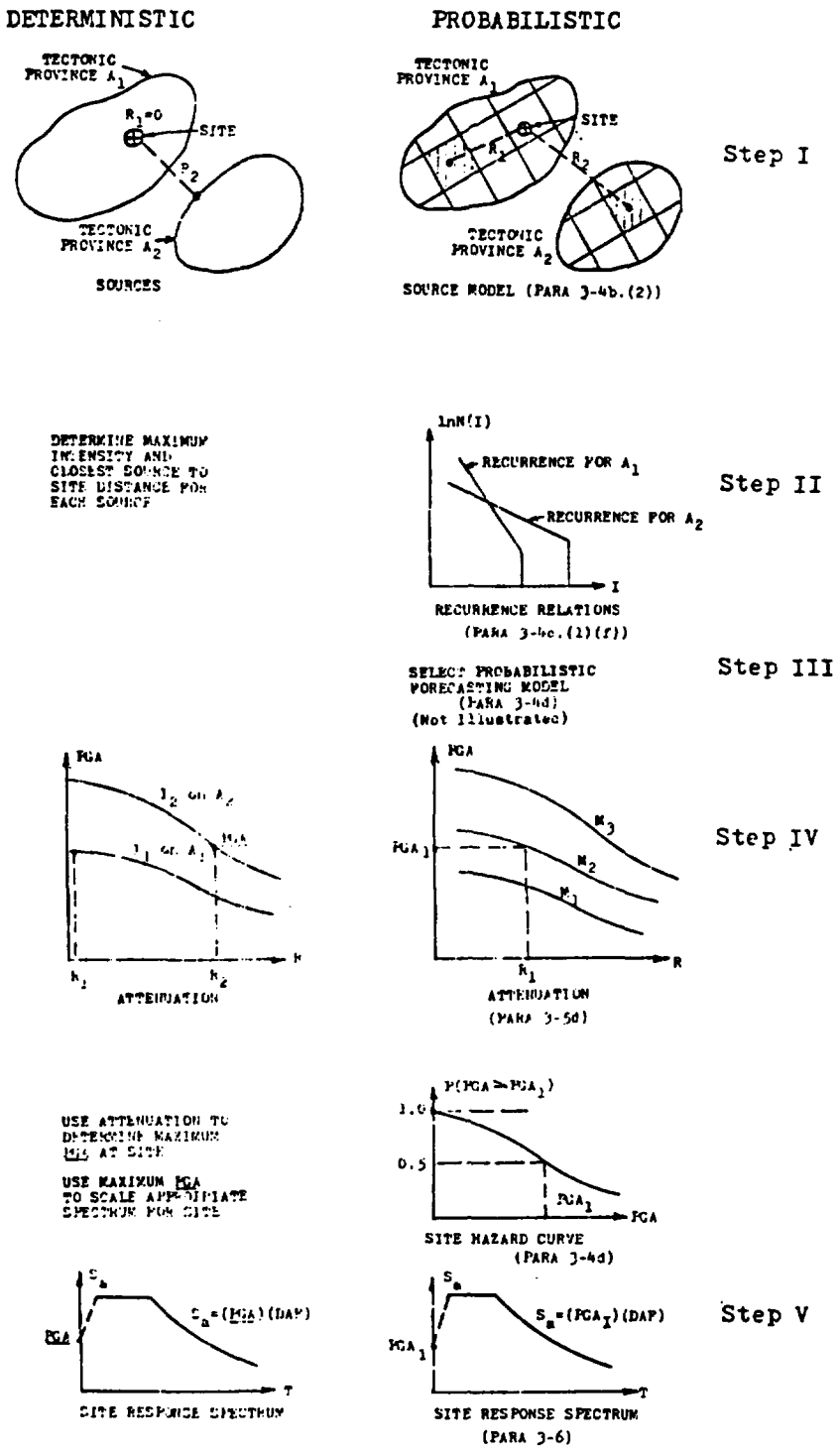

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Figure 3-7. Hazard evaluation of EUS.

one of the magnitude scales (M_L, m_b, M_S) or epicentral intensity I_o, or seismic moment, M_o. The most commonly used size or severity parameter at the source is the Richter magnitude M_L. For the *deterministic approach*, the frequency (or number per unit of time) of occurrences of various magnitude events need not be determined, and the assessment of ground motion at a site will be governed only by the maximum level of earthquake magnitude. For the *probabilistic approach*, the parameters describing the source seismicity must be obtained. This information usually is in the form of a "recurrence relationship," and an upper magnitude or intensity cut-off. The recurrence relationship provides information on magnitude or intensity and the corresponding rate of occurrence or exceedence of that magnitude anywhere on the source under consideration. The upper magnitude cutoff consists of the largest (maximum) possible event that the source can generate. The method of obtaining the above information depends on the type of region and the data base available for the region. See appendix C, paragraph C-1 for background, paragraph 3-4*c* for a detailed discussion and appendix D for examples.

(3) Step III is to project the recurrence information from regional information and past data into forecasts concerning future occurrence. This step is needed in the probabilistic approach only. The forecasting model depends on the type and reliability of the data base. The most commonly used forecasting model is the Homogeneous Poisson probability model. Homogeneous implies a memory-less occurrence of events in time and location. When this homogeneity in time does not appear applicable, Semi-Markov and Markov chain models are used (see Patwardhen et al. (Biblio 50), Vagliente (Biblio 68), Nishioka and Shah, (Biblio 45). These models allow inclusion of memory or time since last event and are more involved and require substantially more information than the Poisson model. A simple extension of the Homogeneous Poisson model, known as the Non-homogeneous Poisson model, may be adapted to incorporate time-dependent information such as the rate of stress build-up and the time since last event, see Savy and Shah (Biblio 52). Another model, usually a uniform probability function, may be employed to represent the random location of event occurrence on the source. See paragraph 3-4*d* for a detailed discussion and appendix D for examples.

(4) Step IV involves the attenuation of the severity parameter from its location on the source to the site. Either intensity or peak ground acceleration for a given magnitude event on the source could be used. The selection of the parameter used for representing the severity and the form of its attenuation relation depends on the region where the analysis is performed and the type of available data. See paragraph 3-5 for a detailed discussion and appendix D for examples.

(5) Step V is to represent the effects of distance, local soil conditions, the magnitude of the seismic event, and the structural foundation size and mass on the frequency content of the ground motion. This is represented by the shape (DAF) of the effective response spectrum for the site and its formulation is described in paragraphs 3-6 and 3-7. The final specified spectrum is of course scaled down by the forecasted site severity. See paragraph 3-8*g* for examples.

b. Use of Results. This available information on ground motion is utilized for design and/or analysis of structures. Chapter 4 shows this utilization for prescribed structural performance and selected risk levels.

c. Selection of Method. The deterministic procedures as outlined in the flow diagrams are used exclusively for those important structures where the consequences of failure are catastrophic; such as nuclear power plants, liquified natural gas facilities, and dams. These procedures tend to compound conservatism (certainty of occurrence, largest magnitude and closest distance from epicenter to the site) and will generally result in extremely large design requirements. For most structures, these highly conservative design values cannot be justified economically for use. This disadvantage of extreme conservatism has actually resulted in the adoption of probabilistic procedures even for some critical facilities. Deterministic procedures, therefore, will not be discussed further in this manual.

d. The STASHA program. The purpose of this manual is to provide the user with an over-all understanding of the procedures, assumptions, and computational methods of ground motion hazard analysis. However, it is most important to recognize that any actual site hazard evaluation would require the use of the computer for development of the various empirical relations and the multiple calculations required for probabilistic accuracy, and prediction uncertainties. In order to perform these calculations in an orderly manner for each step of the hazard analysis, the STASHA Program has been developed by the John A. Blume Earthquake Engineering Center at Stanford University. Both the user's manual and computer program tapes for

STASHA are available at the *Corps of Engineers Office*. In the text of this manual, the STASHA Program will be referenced whenever there is a need for extensive computational effort or for the representative examples contained in the STASHA user's manual (Stanford University, Technical Report No. 36). A description of STASHA and examples are given in appendix D.

Section II. PROCEDURE FOR SITE SPECIFIC GROUND MOTION

3-4. Determination of Source Seismicity.

Each of the probabilistic hazard analysis procedures as presented in paragraph 3-3, and in figures 3-4 to 3-7 is described in this paragraph and in the following paragraphs 3-5 to 3-8.

a. Geotectonic and seismotectonic environment. In the United States, two general regions are defined which are dependent upon the available geologic, geotectonic, geomorphic, historical, and subjective expert information. It will be shown that each of the steps for seismic hazard analysis are region dependent. These regions are the Western United States (WUS) and the Central and Eastern United States (EUS). The boundary between these regions can be defined by the eastern boundary of the Rocky Mountains, (Biblio 5).

(1) *Regional Approaches.* Due to the inherent difference in the geologic structure in the two regions, two major approaches are used in defining seismic sources and assessing future seismic activity. In the Western United States (WUS) and in many other parts of the world, earthquakes occur on faults that extend to the surface of the earth. However, in intraplate regions, such as the Eastern United States (EUS), this is not necessarily true, and it is difficult to recognize and delineate active faults. The two major approaches are (see Biblio 17):

—Active Fault Approach

—Tectonic Province Approach

(2) *Procedure for each approach.* The two regional approaches require different procedures for seismic hazard evaluation. In the active fault approach, seismic sources are relatively well defined along plate boundaries or faults and, hence, the concentration of seismic events and the resulting level of seismicity per unit length of the source or unit area of the source is relatively high. Also, because of the definite location of the source, the source-to-site attenuation distances (R) for the seismic severity parameters are reasonably well defined. In the tectonic province approach, the seismicity is diffused over a large area because no specific faults are identified. Each identified source area is assumed to have homogeneous (uniform) seismicity, and, therefore, the seismicity per unit area is small. However, since the future event could occur anywhere over the tectonic province and, therefore, could be very near the site, the attenuation distances (R) can therefore be short. Also, even though there are considerable variations in seismic severity patterns in the (EUS), these are not as well defined as in the (WUS). There is a general smoothing effect over each entire tectonic province and the boundaries between provinces are often controversial. Also, the relatively low rate of seismic activity in the East makes the recurrence estimation over small areas very difficult. Further, because most Eastern events have occurred in "pre-instrument" times, their source severity data are in terms of the more subjective value of intensity rather than magnitude. Finally, the almost complete lack of strong motion recordings makes the direct empirical development of attenuation relationships in terms of acceleration or velocity impossible. However, both historical reports and seismological studies indicate significantly lower rates of attenuation in the EUS. A summary of regional differences is given in figure 3-8.

b. Source modelling. Step I in seismic hazard analysis is to identify and model seismic sources. This step depends on the following information (see figs 3-9 and 3-12):

—Type and amount of historic seismic occurrence data base.

—Geologic, geotectonic, and geomorphic data base.

—Subjective opinions of experts concerning the seismicity of the region.

The process of source modelling provides two essential portions of information for site hazard analysis:

—First, the configuration of the source and its size establishes the number and location of seismic events for the evaluation of source seismicity in paragraph 3-4*c*.

—Second, the configuration and location of sources relative to the site determines the attenuation distances (R) for ground motion severity in paragraph 3-5.

WESTERN UNITED STATES (WUS)

- Well defined sources
- Significant amounts of data in the form of historical reports, accelerograms, and geological creep measurements.
- Attenuation data in the form of records at different distance and soil conditions.
- Relatively high occurence rates.
- High attenuation of ground motion severity mainly within 100 kilometers.

EASTERN UNITED STATES (EUS)

- Vague description of source provinces.
- Some historical reports, and very few strong motion records.
- Relatively low occurence rates.
- Low attenuation of ground motion severity with significant values at 200 to 300 kilometers.

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Figure 3-8. Regional differences.

It should be mentioned that currently, the USGS researchers are attempting to define seismic source zones for five interior regions of the United States, preparatory to the construction of new national probabilistic ground motion seismic hazard maps. The five regions are the Great Basin, the Northern and Southern Rocky Mountains, the Central Interior and the North Eastern United States (see Biblio 67). Since this work is not yet complete, this manual will develop procedures based on the two regions, the WUS and the EUS. The particular approaches for source identification in each region are described as follows:

(1) *Source modelling in the Western United States.* In this region (see fig 3-9), seismic sources are identified and modelled in the following ways:

(a) Point source. This source characterizes a small region where repeated past earthquakes have occurred. However, no geologically identifiable fault exists. Typically, the size of the region is small compared to the distance from this source to the site. Occasionally,

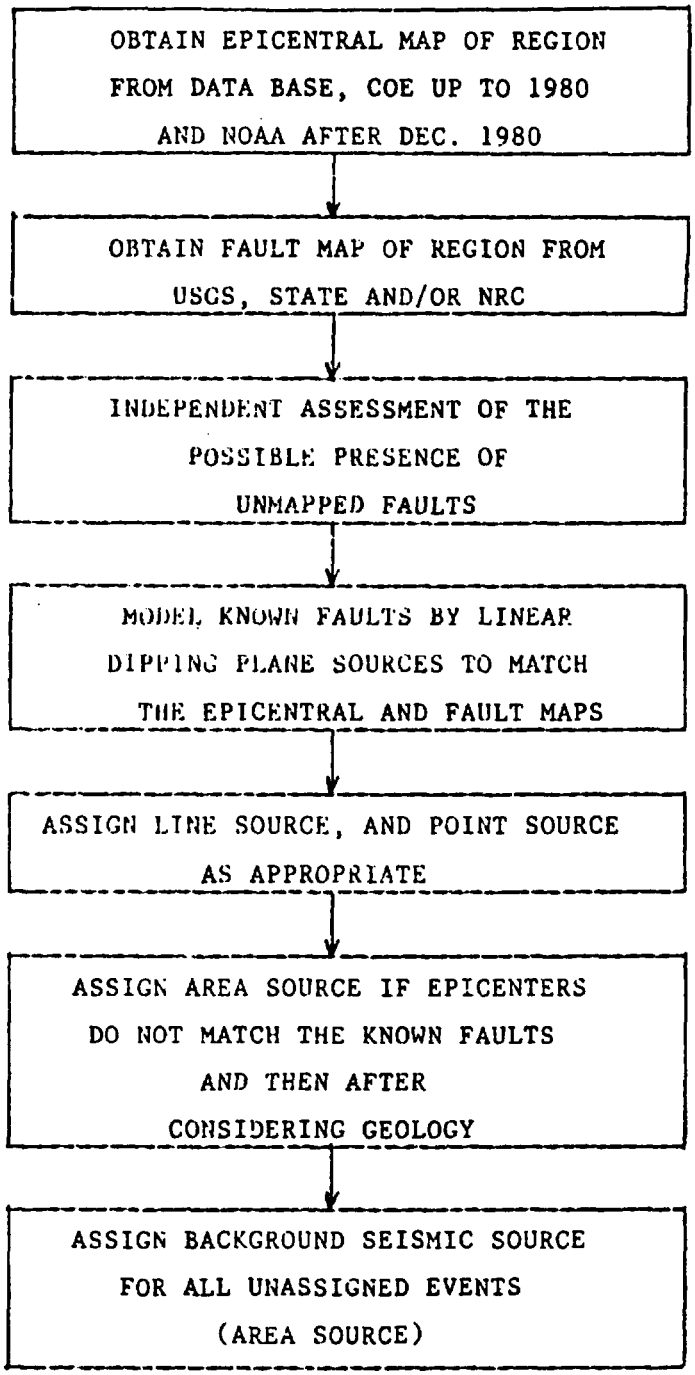

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Figure 3-9. Flow chart for step I source identification and modelling for the WUS.

volcanic sources can be identified as point sources.

(b) Line sources. Fault traces are taken as lines at a certain fixed depth below the ground surface. In California, this depth is usually between 5 to 35 kilometers. This "active fault" modelling approach is used wherever the tectonic structure is more or less evident at the surface.

(c) Area sources. This source model is used when the occurrence of earthquakes in a region cannot be correlated with known faults or the geologic structure of the region. There are also cases where the number of small faults, or a source of clustered activity, may be considered together as an area source.

(d) Dipping plane. This source model is used when one geologic plate thrusts under another plate so as to create a distributed source of earthquakes. This feature is called a Benioff Zone, and can be modelled by means of dipping planes upon which earthquakes have variable epicentral depths. Geological conditions such as this occur in Alaska and in Central America.

(e) Background area source. In general, events that occur somewhat randomly throughout the region and that cannot be associated to any fault or source are treated as background seismicity. They are considered to be part of a large area source with uniformly low seismicity that extends over the area not covered by the other sources. The earthquake location, if not included within one of the previously defined sources, is then in the background zone to account for the possible occurrence of the random or "floating" earthquake. The effect of the background zone is generally small since the contribution of the other sources are governing the hazard. In some particular cases however, where the hazard is low, the background contribution may be non negligible.

(f) Western source conditions. The point, line, and area source models are shown in figure 3–10, and the dipping plane model in figure 3–11. In source modelling, historical records and the knowledge of geotectonic features of the region play an important role. Due to the high seismic activity in the Western United States, and the relatively good geological evidence of faults, surface rupture, and other tectonic features, line sources are used most extensively. Area sources are common in the Pacific Northwest. In regions such as Alaska, both dipping planes and line sources are used: (Biblio 41), and (Biblio 70). An example to demonstrate as to how sources are modelled is given in appendix D.

(2) *Source modelling in the Eastern United States.* In the Eastern United States, the tectonic province approach is used (see fig 3–12). There are various reasons for adopting such an approach; the most important being that the degree of fault and seismic activity in the Eastern United States is low, resulting in very little geologic and historic evidence. Also, in large areas of the Eastern United States, there is a scarcity of geologically recent deposits that would record evidence of recent fault activity. In addition, the heavy vegetation covers the faults and prevents their detection. Finally, the recent developments of evaluating fault activity in the (WUS) have not been applied in the east due to excessive cost and time involvement; except in a few regions such as New Madrid where faulting evidence has been substantiated (Biblio 71).

(a) Area source configuration. One of the key features of tectonic province approach is to delineate these provinces as area sources that have a uniform potential to generate earthquakes. Within that area, the future earthquake activity should be homogeneous. Due to lack of sufficient historical and geological evidence, there is no unique and generally consistent way of delineating these area sources. Two examples on area source configurations for the Eastern United States are shown in figures 3–13 and 3–14.

(b) Using subjective input as furnished by interviews from ten experts, Mortgat (Biblio 63 and 64), has developed homogeneous area sources as shown in figure 3–15. With respect to this method of using expert opinion, it is well to recognize that experts form their objective biases from the particular data and other geologic and seismologic evidence that they may have seen. Since most of the experts work with a similar data and information base, the variability in their individual source configuration is due to their personal biases. Barstow et al (Biblio 5) have studied statistical techniques to provide a methodology for the production of working tectonic province and tectonic structure maps for the Eastern and Central United States, identifying areas of uniform seismic hazard.

c. Source seismicity. Step II in seismic hazard analysis is to evaluate the seismicity of each of the modelled source (see fig 3–16). Evalua-

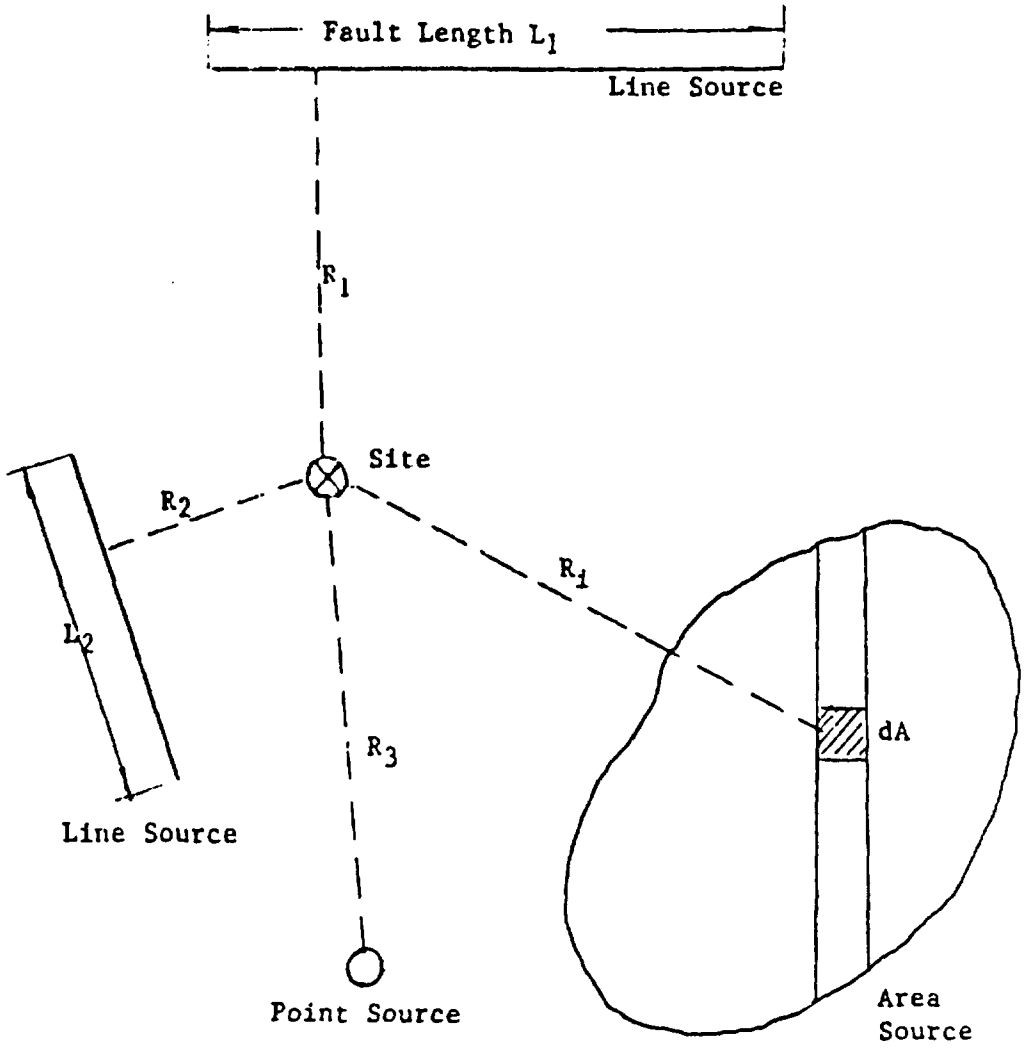

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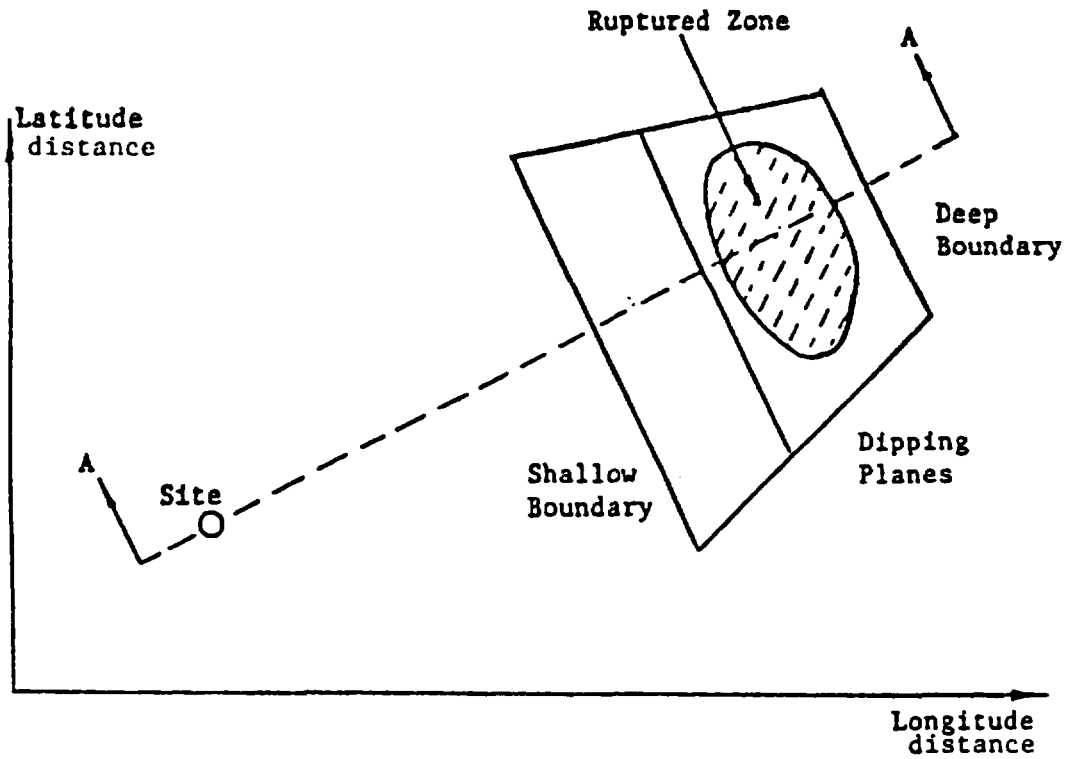
Figure 3-10. Point, line and area sources.



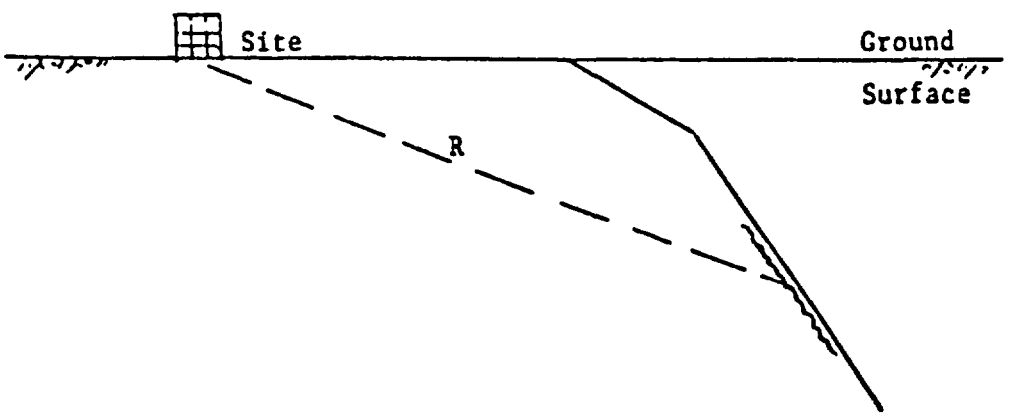


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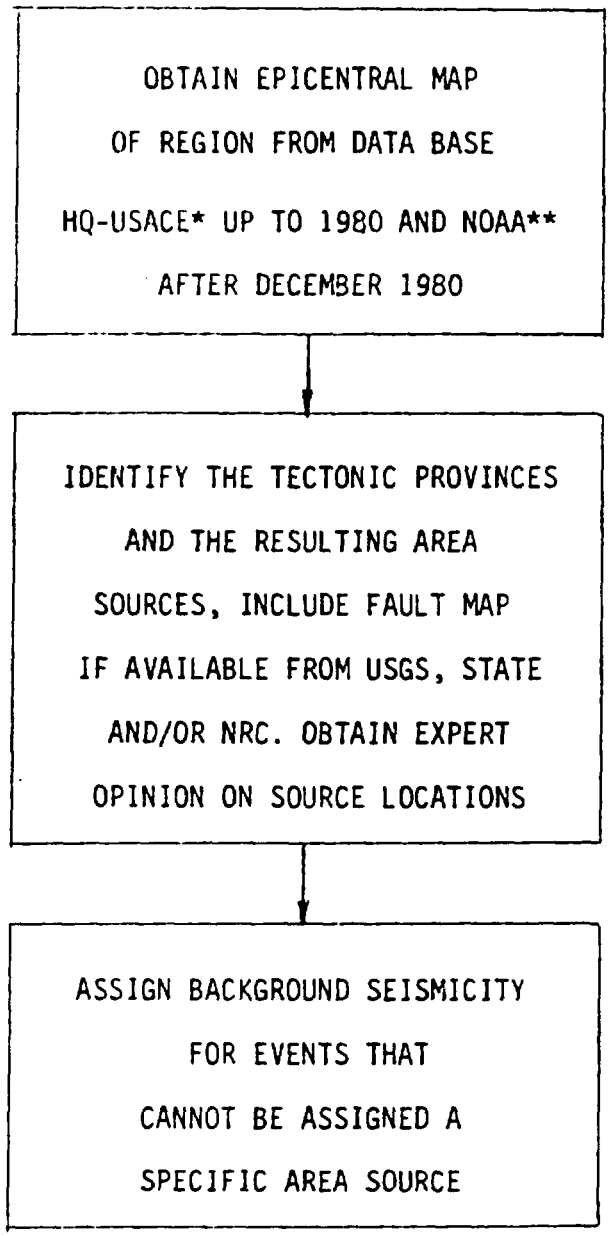
Figure 3-11. Dipping plane source.



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325 Broadway, mail code D-623
Boulder, CO 80303

NOTE: If at a future date, specific faults are identified, then they can be modeled by means of line or dipping plane sources.

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Figure 3-12. Flow chart for step I source identification and modelling for the EUS.

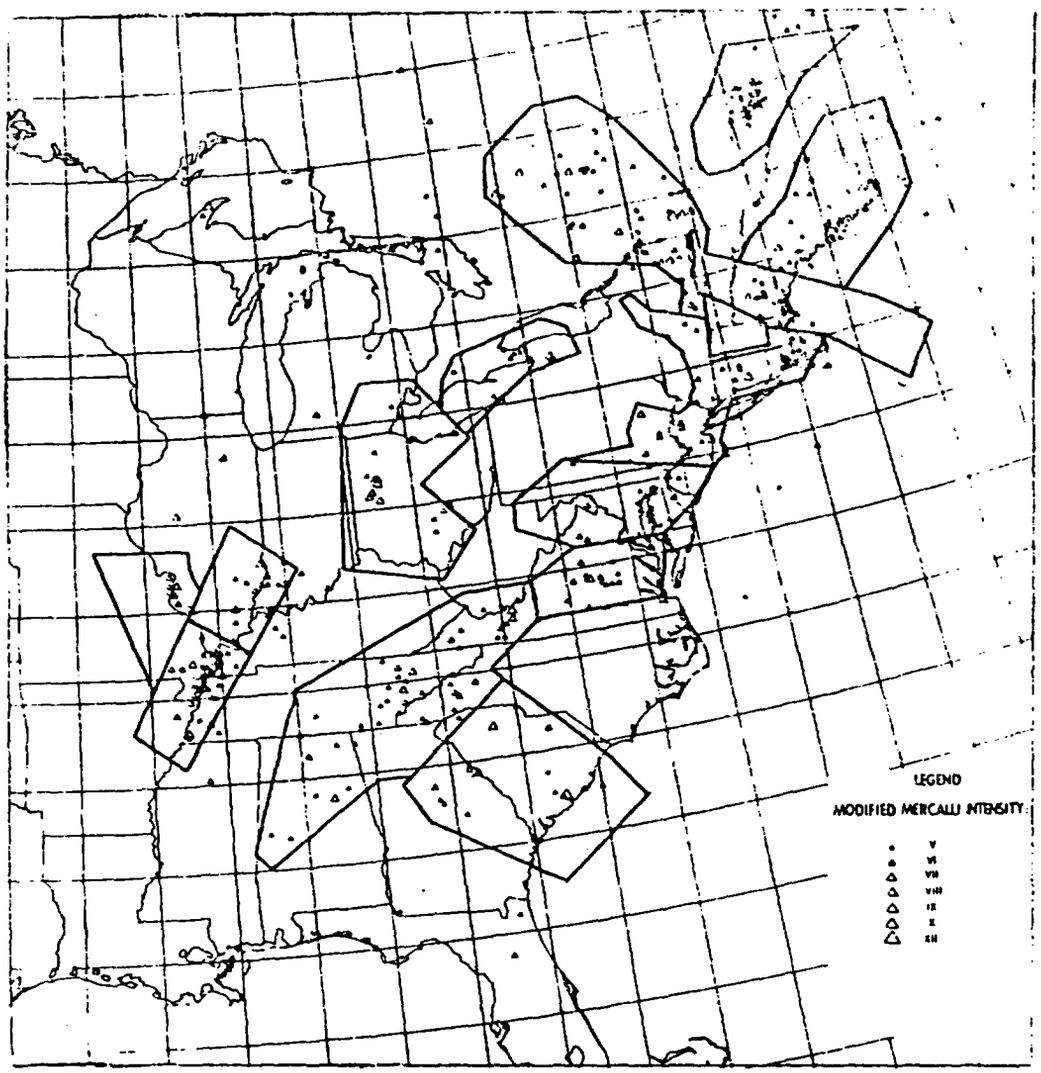

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Figure 3-13. Seismic sources after Algermisson and Perkins (1976).

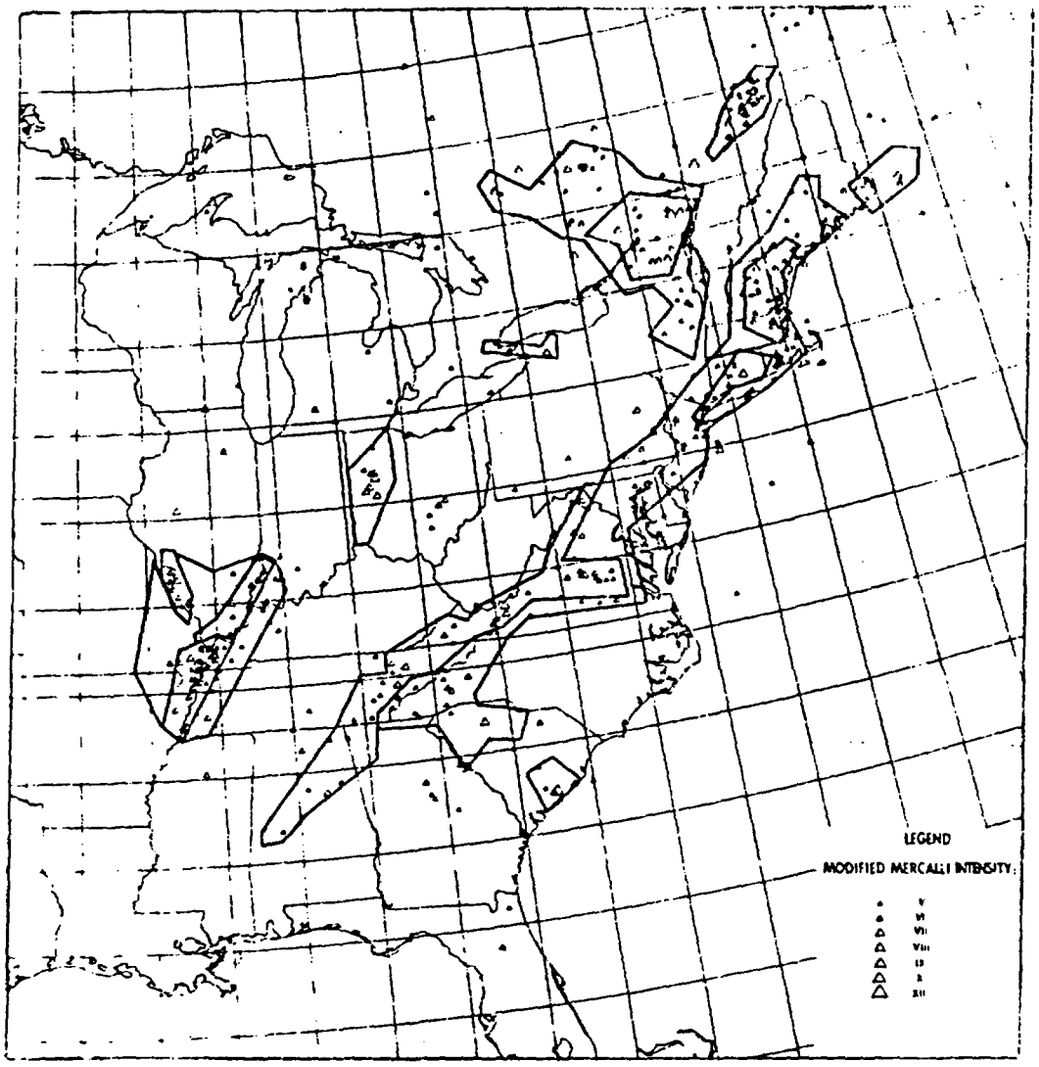

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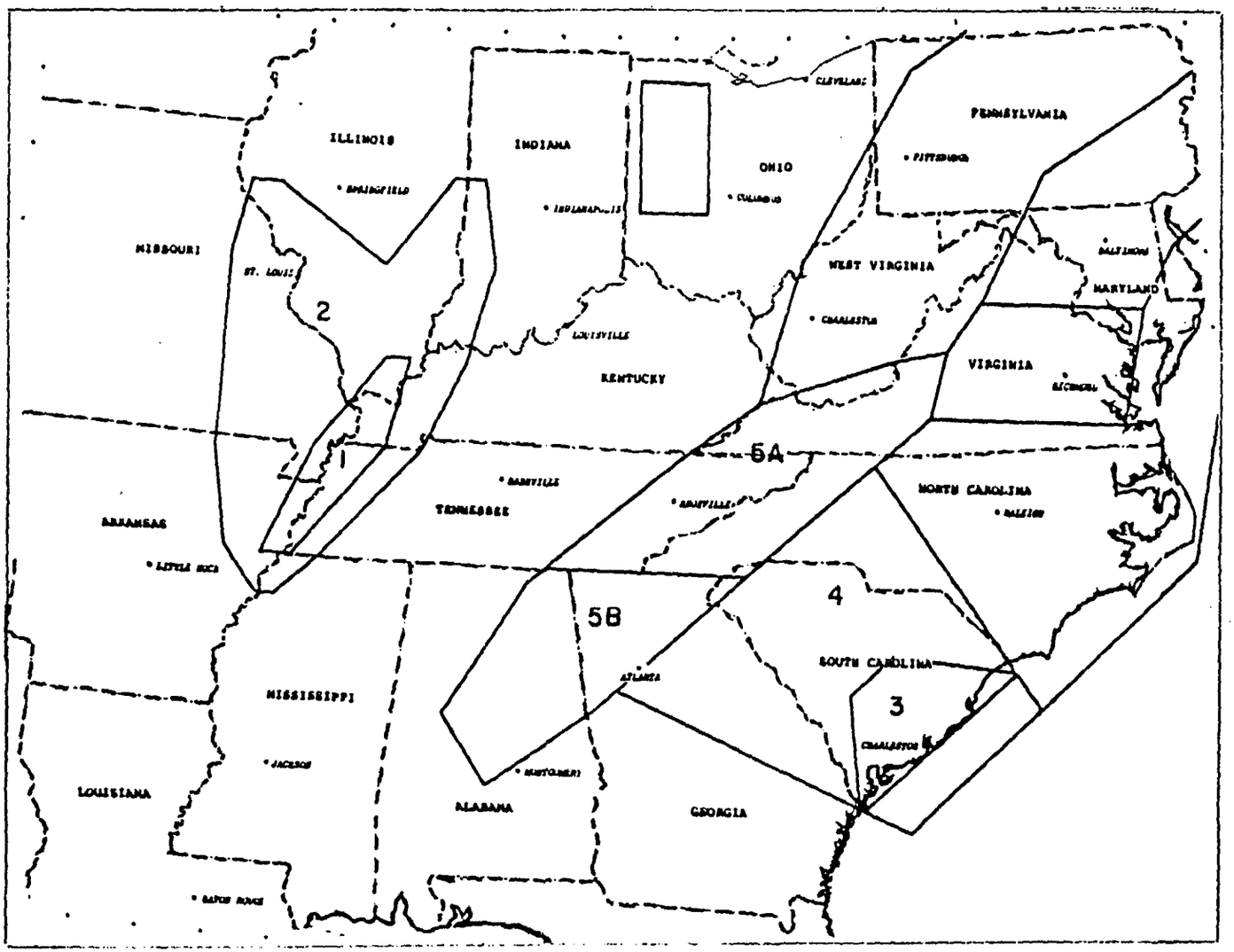
Figure 3-14. Seismic sources after Hadley and Devine (1974).



Reprinted from "Seismic Hazard Analysis-Solicitation of Expert Opinion," Nuclear Regulatory Commission, Tera Corporation, NUREG/CR-1582, Vol. 3, 1980.

Figure 3-15. Seismic source after Tera (1981).

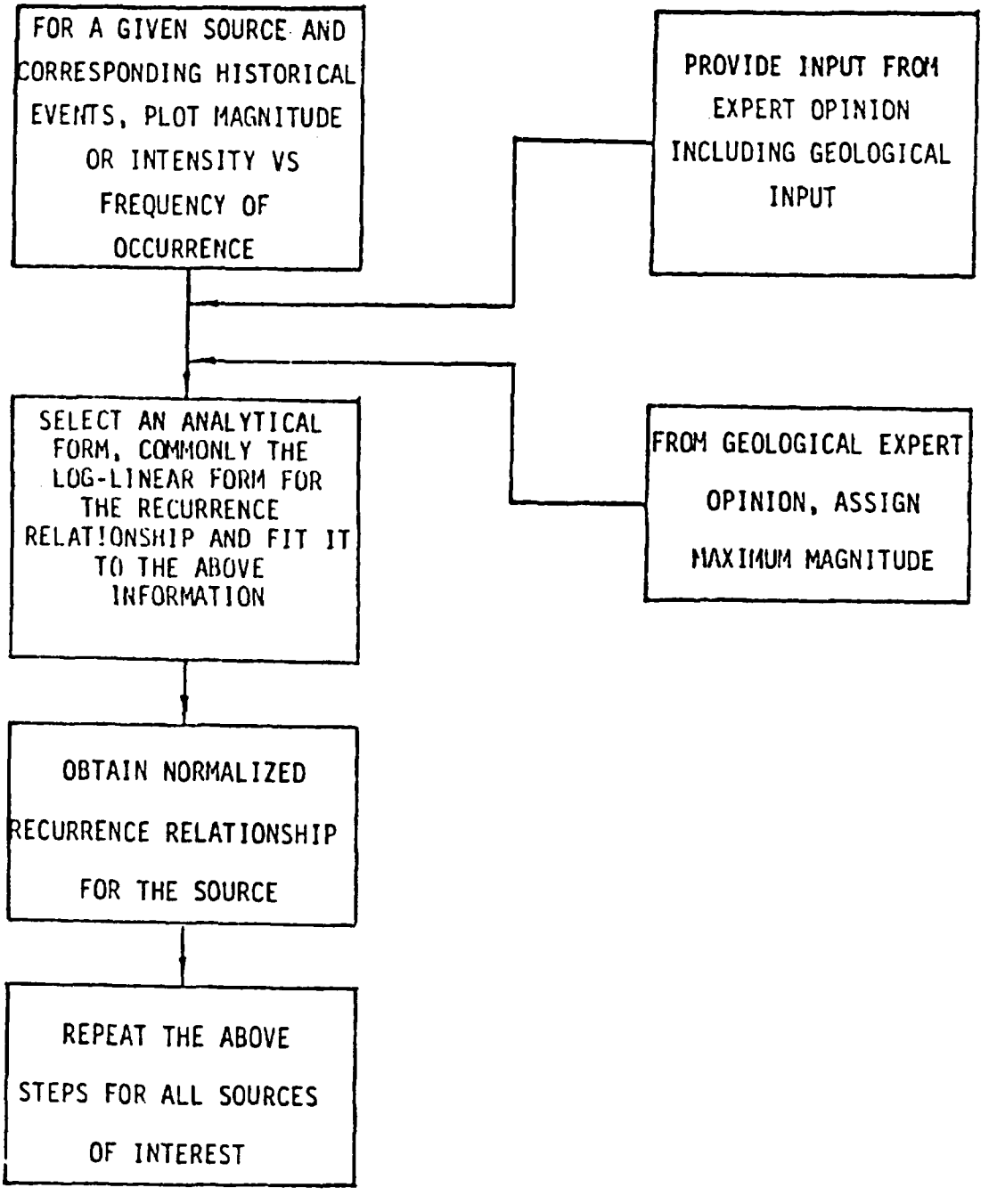


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Figure 3-16. Flow chart for step II source seismicity and recurrence relationship for WUS and EUS.

tion of seismicity involves the following components:

—Collection and processing of occurrence data and formulation of the recurrence relationship.

—Determination of the size of the maximum earthquake a given source is capable of generating.

(1) *Collection of data and formulation of the recurrence relationship.* The data base for seismic events on a given source is often incomplete, nonhomogeneous in time, and lacking in refinement. The appropriate processing of this occurrence information is very important because the reliability of results of the hazard analysis are strongly dependent on the consistency and the completeness of the input data base. The magnitude-frequency or recurrence relationship is formulated from the number of earthquakes that a source has generated and their respective magnitudes. The most common method of determining this relationship is from historic data. Occasionally, other information sources, such as geological evidence and slip rate of the fault, are used to supplement this historical data base. Statistical regression analysis is commonly used to obtain the best line fit with the "least squared" error. Expert subjective opinion can also be incorporated in order to supplement the historical data base. The most commonly used magnitude-frequency relationship is the one suggested by Gutenberg and Richter (Biblio 26). In this relationship, the source severity parameter could be either magnitude or epicentral intensity. The type of parameter and the constants of the magnitude-frequency relationship vary from one region to the other. Data adjustment is usually necessary before using the data to determine the parameters of the magnitude-frequency relationship. It has been observed that the completeness of earthquake records varies with time. In the past, due to low population density and lack of interest in earthquake activity, only large events were recorded. With increased instrumental coverage, intermediate and lesser earthquakes have been recorded with more frequency, producing an apparent increase in seismic activity with time which biases the statistics from uncorrected catalogs of data. In recognition of this time bias, the evaluation of the degree of completeness of the available earthquake record is an important step in the analysis of data. One possibility is to confine analysis to sections of the record that are complete for the earthquakes of interest. The main problem with this approach is that it reduces the size of the useful sample and meaningful statistical averages of large earthquakes cannot be obtained because of their infrequent occurrences (Biblio 6). An alternative is to correct for incomplete reporting by a random simulation of missing data (see STASHA). The Gutenberg-Richter relationship is given by equation (3-1).

$$\ln N(m) = \alpha + \beta m \qquad \text{(eq 3-1)}$$

where

ln = Natural log to the base e

N(m) = Average Number of events greater than or equal to the magnitude m.

α, β = constants.

Very often, this relationship is used in a slightly different format where logarithm to the base 10 is used instead of to the base e.

$$\log_{10} N(m) = a + bm \qquad \text{(eq 3-2)}$$

One would convert the equation from base e to base 10 by means of the following simple conversion:

$$a = 0.43429\alpha \qquad \text{(eq 3-3)}$$

$$b = 0.43429\beta \qquad \text{(eq 3-4)}$$

Such magnitude-frequency relationships are called "recurrence relationships" in the literature and a general example is shown in figure 3-17. After the recurrence relationship is obtained, the following normalization process can be performed.

(a) Normalization to unit length and time. Let T be the time-period over which the recurrence data has been obtained. If the source is a line source, let L be the length of this source. Then, N(m) = average number of events equal to or greater than magnitude m during the time period T and on source length L for the line source.

Let

$$N'(m) = \frac{N(m)}{LT}.$$

then

$$\ln(N'(m)) = \ln\frac{N(m)}{LT} = \ln N(m) - \ln(LT)$$

$$\ln(N'(m)) = \alpha + \beta m - \ln(LT)$$
$$= \alpha - \ln(LT) + \beta m$$

or

$$\ln(N'(m)) = \alpha' + \beta m \qquad \text{(eq 3-5)}$$

where

$N'(m)$ = average number of events equal to or greater than magnitude m per unit time and unit of source length

$$\alpha' = \alpha - \ln(LT)$$

Note that the value of β does not change when the recurrence relationship is normalized. This step of normalizing the recurrence relationship is usually done by the seismic hazard analysis computer program. The purpose of presenting this step here is to indicate that in the normalization, it is assumed that for a given source, the number of events equal to or greater than a given magnitude is homogeneous in time and space. Thus, the mean rate of occurrence does not change with time or along the given source. More will be discussed on this topic when the probabilistic-forecasting models are presented.

(b) Normalization to unit area and time. If the area source with area A was considered instead of the line source, the relationship would have a simlar format:

$$N'(m) = \frac{N(m)}{AT}$$

$$\ln(N'(m)) = \alpha - \ln(AT) + \beta m$$

or

$$\ln(N'(m)) = \alpha' + \beta m, \quad \text{with } \alpha' = \alpha - \ln(AT) \qquad \text{(eq 3-6)}$$

Where $N'(m)$ and α' are now normalized with respect to the source area A.

(c) Sampling uncertainty. For a given magnitude m, the fitted line gives the average value of $N(m)$ or $N'(m)$, and this average or expected rate value is required for the probabilistic forecasting model in paragraph 3-4*d*. However, there is considerable scatter of the actual recorded number of events. To take this scatter into account, a probability distribution function is generally assumed for the number of events equal to or greater than a given m. Further, the fitted recurrence line, because of limited data base and the largely subjective evaluation of the maximum magnitude, has a sampling error. This sampling error is an indicator of the difference between the sample fitted line from the limited data source and the true line that would be obtained from a very large data source, figure 3-17. The STASHA, (Stanford University, Technical Report No. 36) program gives a probabilistic representation for this sampling uncertainty in the $N(m)$ value.

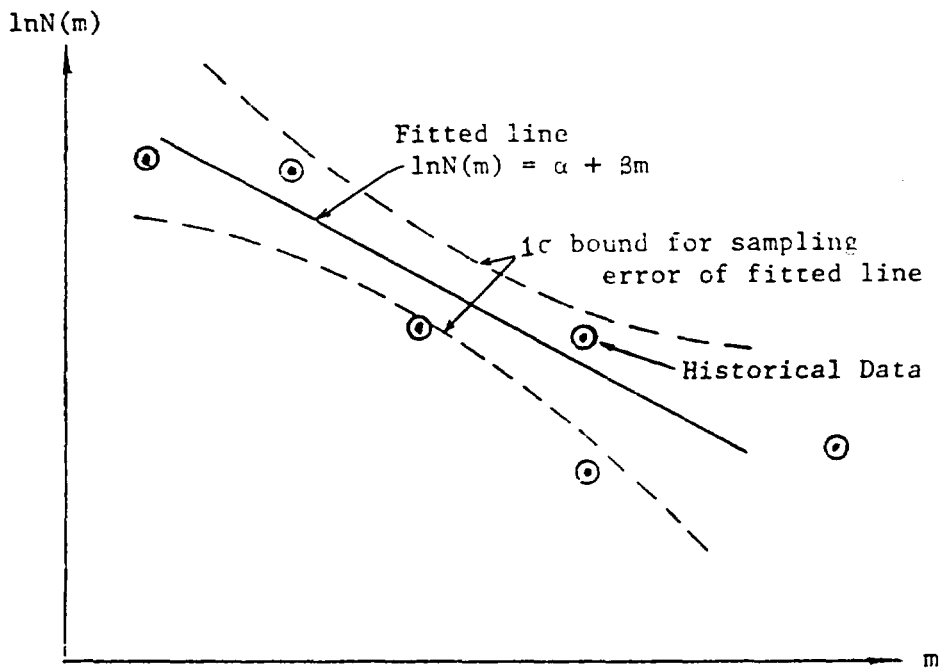

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Figure 3-17. Linear Recurrence Relationship.

(d) Non-linear relations. Other forms of recurrence relationships have been used by researchers. Dalal (Biblio 21) has used Gaussian and log-Normal probability distribution models. Mortgat et al (Biblio 42) have used a bilinear relationship as shown in figure 3–18. Here, two lines are fitted to the data. The point where the two lines meet is usually determined subjectively from the geologic considerations concerning capabilities or rates of large magnitudes on the source. Cornell and Merz (Biblio 39) have used a quadratic form for their recurrence relationship. Recently, Dong et al. (Biblio 23) have applied the maximum entropy concept to obtain minimally biased recurrence relationships. (See app D for some examples).

(e) Recurrence relationship for sources in the Western United States. The "active fault" approach is usually employed in this region. Therefore, based on the fault locations and the modelling of these faults as line sources, past seismic events are assigned according to their relative proximity to the different sources. This process of event assignment is usually performed by expert judgement with recognition that epicentral locations are subject to error and that events are more likely to occur on the known fault rather than on the adjacent area. The STASHA program has a procedure for event assignment. It has been found that the value of the recurrence constant for most of the WUS sources lies between about 1.1 to approximately 2.5. Figure 3–19 shows the recurrence relationship for the northern section of the San Andreas Fault in California. It should be mentioned here that one large fault such as the San Andreas may be broken down into two or more homogeneous segmental sources and the recurrence relationship may be determined for each of these segmental sources. This use of homogeneous segments is quite common in California where there is evidence of varying degrees of seismicity on the large sources. The source severity parameter employed in developing these recurrence relationships in the WUS is usually the Richter magnitude (which can be considered to be the same as the local magnitude M_L). In appendix C, paragraph C–1, these variatious magnitude scales are defined.

(f) Recurrence relationships for sources in Eastern United States. The tectonic province approach is used for modelling sources in the eastern United States. Therefore, all the sources are area sources, and these usually cover rather large regions. With respect to the source severity parameter, most of the historical data in the East is compiled in the form of the Modified Mercalli Intensity Scale. However, there are cases where the most recent data is in local Magnitude (M_L) or body wave magnitude (m_b).

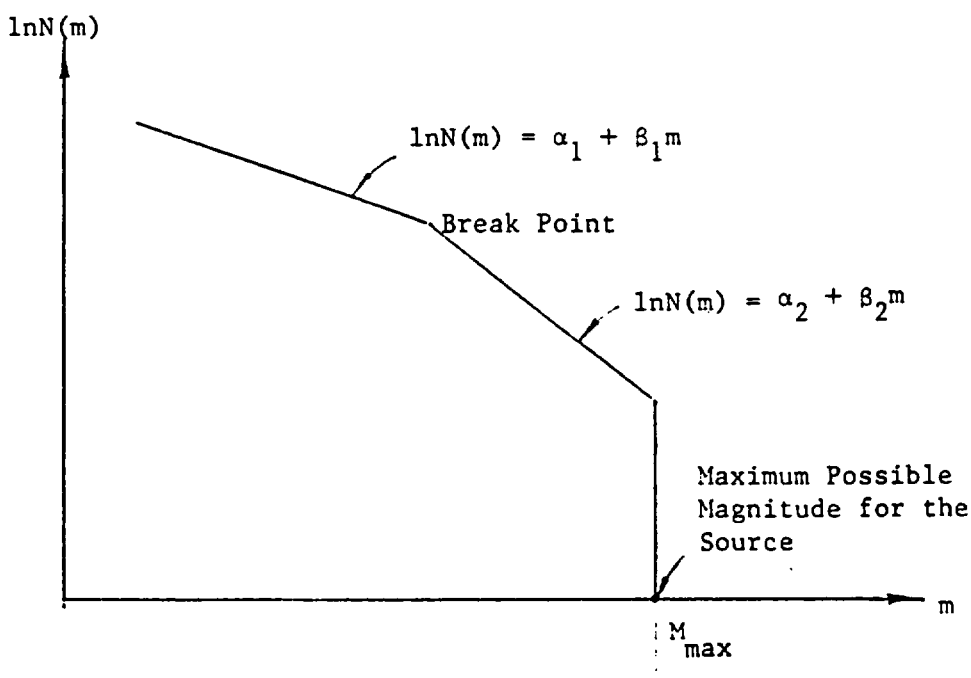

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Figure 3–18. Bilinear Recurrence Relationship.

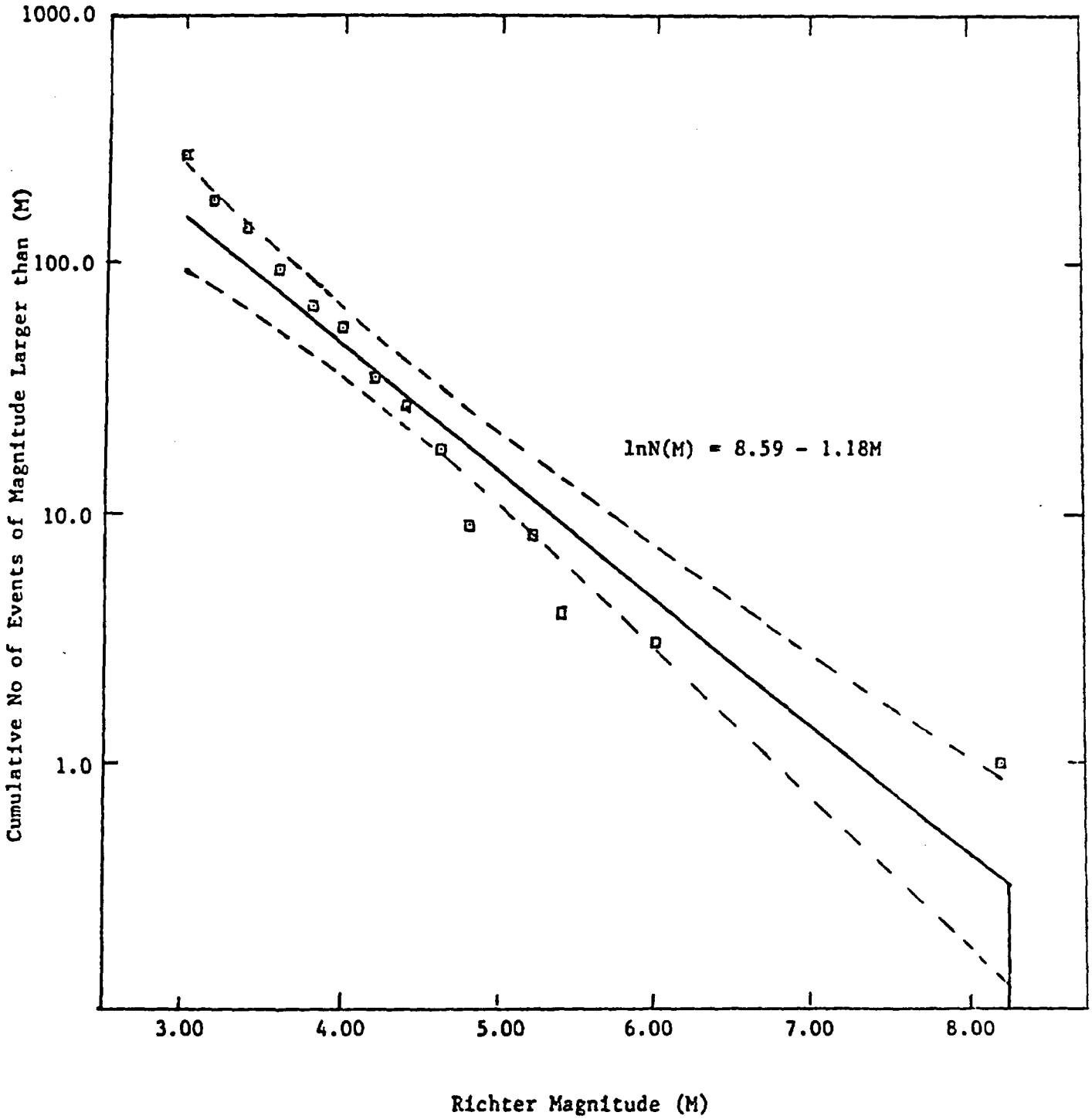

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Figure 3-19. Recurrence relation for North San Andreas

Therefore, in order to make all the data consistent, one approach consists in converting the magnitude into an epicentral Modified Mercalli Intensity (MMI).

Let I_o be the epicentral MMI

M be the Richter magnitude.

In paragraph C-1, appendix C, the relationships between these two parameters for the Eastern United States are given.

Also, Nuttli (Biblio 46) has developed a relationship between the body wave magnitude m_b and the epicentral intensity I_o,

$$m_b = 1.75 + 0.50I_o \qquad \text{(eq 3-7)}$$

Using relationships such as these, the occurrence data in the form of the source intensity I_o can be obtained. The Gutenberg-Richter recurrence format for intensity is then written in the following form:

$$\ln[N(I_o)] = \alpha_I + \beta_I I_o \qquad \text{(eq 3-8)}$$

Where α_I and β_I are regression constants. Yegian (Biblio 71) and TERA (Biblio 63,64) have given values of β_I for the EUS. A shortcoming of using epicentral intensity (I_o) as a parameter is that I_o, unlike magnitude, is not a direct measure of a source severity. By definition, intensity is a number corresponding to particular observed effects and these are often influenced by both the site condition and the prevailing local types of construction. In order to overcome this shortcoming, an alternative approach involves the estimation of magnitude of the historical events (before instrument records) in terms of their estimated epicentral intensity, felt area, and fall-off intensity. This requires a large amount of background research effort. However, most large events in the EUS have been assigned a magnitude based on this method by different researchers (Nuttli, et al. (Biblio 47)). Smaller events of less importance in the analysis can be converted to magnitude using one of the relationships in appendix C-1, or equation 3-7. In the formulation of the recurrence relation in the EUS, it is usually assumed (because of lack of data) that the same β_I value applies throughout very large regions and that local variations apply only to the level of seismicity (parameter α_I). The range of values for the parameter β_I is from 0.80 to 0.92.

(2) *Determination of the maximum earthquake.* One of the most controversial and important variables of interest in representing source seismicity is that of the size of the maximum earthquake. Past literature has employed the term "maximum credible" event. Such a term should be discouraged from use. Instead, use of a term such as the "maximum earthquake size" should be encouraged. The size of the maximum earthquake is used in source seismicity determination in two ways:

—Deterministic use of the maximum earthquake in the design process (see figures 3-4 and 3-6).

—Probabilistic use of the maximum earthquake, in the recurrence relationship. Here, the value of this earthquake size provides the upper cut off magnitude in linear recurrence relationship, or it could be an asymptote in the non-linear recurrence relationship, see figure 3-18.

The estimate of the size of the maximum earthquake for a given source is based on the following factors:

1—Geologic evaluation of the regional tectonic framework.

2—Historical seismicity of the source and the surrounding region.

3—Geologic history of displacement (from trenching investigations).

4—Relationship between earthquake magnitude and fault rupture length.

5—Relationship between earthquake magnitude and amount of fault displacement.

Out of the five factors mentioned above, the tectonic province approach in the EUS would permit the use of only the first three. When the active fault approach is employed in the WUS, then all of the five factors will be used for such an evaluation. Whether one decides to use a specific maximum earthquake value or a probabilistic distribution representation of the maximum earthquake value, the STASHA program can handle both forms of this input information.

(a) Determination of the Size of the Maximum Earthquake-Western United States. In this region, seismic sources are usually line sources (active fault approach). For such sources the maximum earthquake size is usually based on the fault rupture length or the maximum amount of displacement that may be associated with the causative fault. Not only the historical data base is used, but also geological data from trenching or other geomorphological studies; Sieh, (Biblio 60) can be employed. Recently, (Aki, (Biblio 1); Kanamori and Geller, (Biblio 24); Molnar, (Bib-

lio 40)) seismic moment has been related to the fault rupture area, along with the fault shear modulus and average slip. Relating the maximum seismic moment $M_{o,max}$ to moment magnitude M_m gives the value of the largest moment magnitude. It is useful to note that M_m is equal to M_L for M_L values between 5 and 7. Empirical relationships between M, fault rupture length L and fault displacement D are developed from world wide data (Bonilla and Buchanan, (Biblio 11); Slemmons, (Biblio 61)). Paragraph C-4, appendix C gives these relationships. The tables and relationships presented in paragraph C-4 should not be used exclusively but together with historical and other geologic evidence. The historical record of earthquakes in a given region may be one of the few indicators of the potential for future earthquakes. However, extreme caution must be exercised when extrapolated forecasts are made. The time period of records in the United States is relatively short and therefore statistical prediction should always be compared or modified by expert judgment concerning seismicity. In paragraph C-4, table C-11 shows the slip rate activity of some of the faults of the Western United States, and figure C-10 shows fault slip versus time. This type of information can also be incorporated probabilistically in assessing fault activity and in estimating the size of maximum earthquake events. This will be discussed further in the forecasting paragraph, 3-4*d*.

(b) Determination of the Size of Maximum Earthquake-Eastern United States. In this region, seismic sources are modelled by the tectonic province approach. The most commonly used method of determining the size of the maximum earthquake is through historical records. Very little information (if any) is available on the fault rupture or fault displacement and hence these two parameters cannot be related to the size. To overcome the problem of limited historical data in estimating the maximum earthquake size, the opinions of experts should be obtained. Two principal methods are used to determine the maximum earthquake size. The first one consists of using the size of the largest historical event subjectively incremented by a safety factor such as half a magnitude or one intensity unit. The other consists in using the earthquake size corresponding to a 1000 to 5000 year return period from the recurrence relationship. Although this last method is somewhat ad hoc it is felt that, in the present geologic framework, the near future will be similar to the past and that the 1000 to 5000 year choice represents a low enough probability such that the corresponding event can be considered as an upper bound. This last approach should include all available information such as local or regional strain release or stress field data (See para 3-4*c*(3)).

(3) Use of Seismic Moment to Represent Source Seismicity. One of the more recent developments in seismic hazard analysis is to use seismic moment (M_o) to describe source seismicity. Seismologists have introduced a "physical" parameter called seismic moment M_o to describe size of an earthquake. This development is relatively new and its practical implementation for seismic hazard analysis has not been achieved. Paragraph C-4, appendix C, introduces the users of this manual to this new concept.

d. Probabilistic Forecasting Models. Step III is to forecast source severity of future earthquakes on each of the identified sources (see fig 3-20), once the sources of seismic activity have been identified (para 3-4*b*) and the seismicity of the identified sources has been determined (para 3-4*c*). These forecasting models are not based on extrapolation of past data, but are based on stochastic models. These models from the probability theory field of stochastic processes may however employ data for the evaluation of their parameters. The type of stochastic forecasting model selected depends on the acceptable type and level of assumptions about the seismic occurrence on each of the sources. The most widely used model is called the homogeneous Poisson Model. Typical examples of this approach are given in the following references: Cornell (Biblio 18), Cornell and Van Marcke (Biblio 19), Stepp (Biblio 62), Algermissen (Biblio 3), McGuire (Biblio 37), Shah et al. (Biblio 58), Wiggins (Biblio 69), Der Kiureghian and Ang (Biblio 22), Liu and Fagel (Biblio 34), Kiremidjian and Shah (Biblio 32). This is normally called a memoryless process because of the assumption that the probability of occurrence or nonoccurrence of an earthquake in any given year and for a given source does not depend on the time interval since the last occurrence. For most practical cases where the future time horizon is of the order of fifty to one hundred years, this is a reasonable assumption and is suitable for the purposes of this manual. A non-homogeneus Poisson model has also been used to account for the dependence of the mean rate of occurrence on time. Savy and Shah 1981, (Biblio 52) have shown the use of this model. In order to account for the lack of sufficient historic occurrence data and also to take into account geological data (such as slip rate, size of past rupture

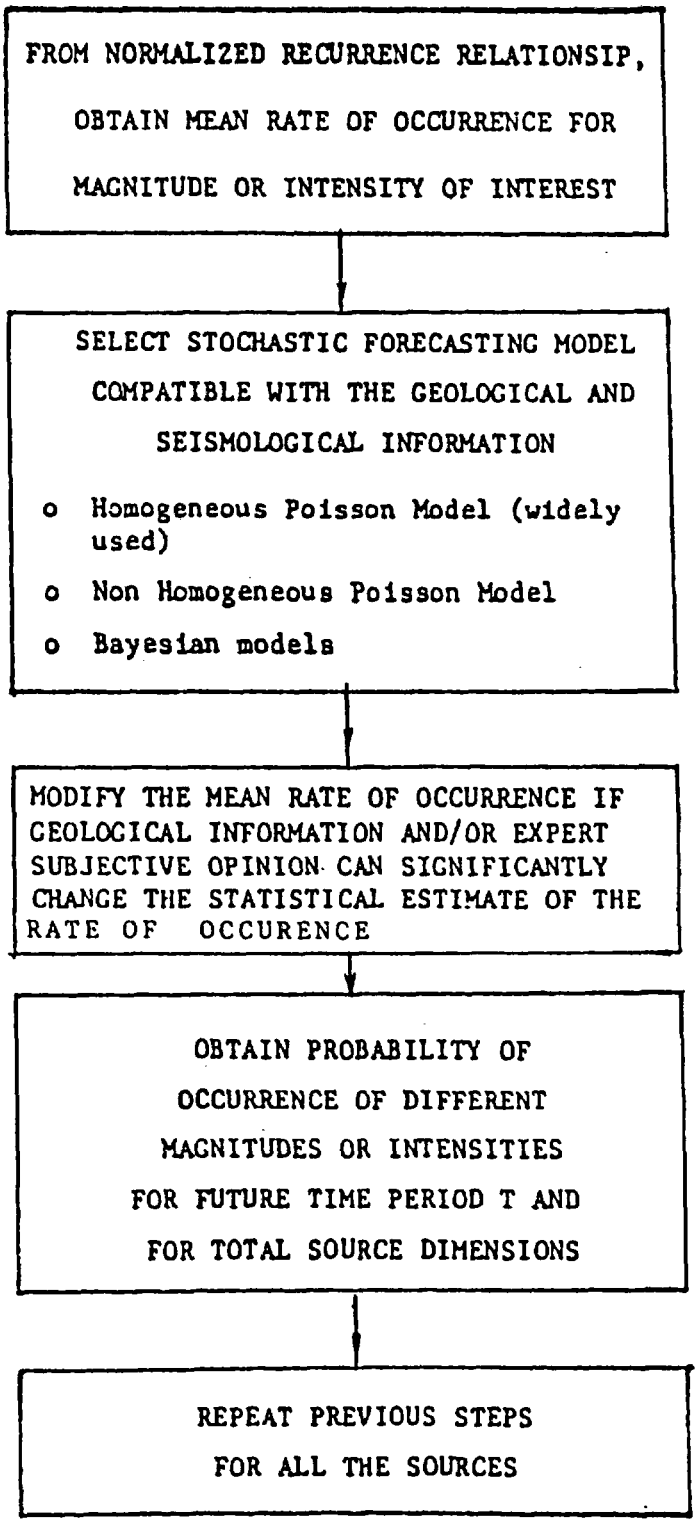

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Figure 3-20. Flow chart for step III seismic forecasting model.

length and the amount of fault displacement per event), Bayesian models have been developed. These models assume a Poisson occurrence model along with a Bernoulli model for the size of each occurrence. The STASHA program describes this type of model; see appendix D for an example.

(1) When the occurrence of a future event is independent of the past occurrences, then the homogeneous Poisson model is a reasonable model. The Poisson model of occurrence can be written as

$$P_N(n,t) = \frac{e^{-\lambda t}(\lambda t)^n}{n!} \qquad \text{(eq 3-9)}$$

where $P_N(n,t)$ = Probability of having n events in a future time period t

n = number of events

λ = mean rate of events per unit of time (years)

(2) If λ is independent of time, then the process is called homogeneous. If λ varies with time, the process is called non-homogeneous.

(3) For earthquake events to follow the homogeneous poisson model, the following assumptions must be valid:

—Earthquakes are spatially independent;

—Earthquakes are temporarily independent;

—The probability that two seismic events will take place at the same place and at the same instant of time approaches zero.

The first assumption implies that occurrence or nonoccurrence of a seismic event at one site or location or source does not affect the occurrence or nonoccurrence of another seismic event at some other location or source. The second assumption implies that the seismic events do not have memory in time. The third assumption implies that for a small time interval dt, no more than one seismic event can occur. This assumption is considered to be realistic and fits the physical phenomenon reasonably well.

(4) It can be shown that if the arrival of earthquake events follow the Poisson process, then the random description of the time interval between two events follows exponential distribution. Thus,

$$f(t) = \lambda e^{-\lambda t} \quad t \geq 0 \qquad \text{(eq 3-10)}$$

$$= 0, \text{ Otherwise}$$

$f(t)$ is the probability distribution function for the interarrival time t between events, and

λ is the mean rate of occurrence.

If one defines the return period (T_R) as the time interval during which the expected number of occurrences is one, then this much used engineering parameter in risk analysis is obtained as follows: the expected number of events for the Poisson process of equation 3-9 is given by

$$E(N(t)|\lambda) = \lambda t \qquad \text{(eq 3-11)}$$

where $E(N(t)|\lambda)$ = Expected number of events for future time t given λ.

If equation 3-11 is equated to one, we get the definition of return period.

$$\lambda T_R = 1$$

and hence

$$T_R = \frac{1}{\lambda} \qquad \text{(eq 3-12)}$$

T_R is therefore the average time interval between events, and is also the reciprocal of the annual risk of occurrence. The value of λ is usually obtained from the recurrence relationship developed in paragraph 3-4c. Let $N'(m) = \alpha' + \beta m$ be the average number or rate of events equal to or greater than magnitude m per unit of time and per unit of source dimension. Then, using the Poisson occurrence model, the probability of n events equal to or greater than magnitude m in future time t for source of length L (or area A) is given by

$$P(n,m,t) = \frac{\exp(-N'(m)Lt)^n (N'(m)Lt)^n}{n!} \qquad \text{(eq 3-13)}$$

Thus,

$$P(O,m,t) = \exp(-N'(m)Lt) \qquad \text{(eq 3-14)}$$

or probability of at least one event above magnitude m for a source of length L in future time t is given by

$$1 - P(O,m,t) = 1 - \exp(-N'(m)Lt) \qquad \text{(eq 3-15)}$$

Equation 3-15 provides the most elementary hazard statement for the occurrence of a given magnitude (or greater) on a given source. The probability of exceeding a given level of site intensity (such as PGA) needs consideration of the location of the event (epicenter or rupture length) on the source and also the consideration of all sources affecting the site. This is treated in the next paragraph 3-5.

3-5 Selection of the attenuation relation for the determination of seismic severity at a site.

Step IV of the seismic hazard analysis deals with the methods of evaluating the severity of ground motion at the site where the structure is located, given the information developed in the previous three steps.

a. Attenuation of ground motion. When a rupture along a fault plane occurs, vibratory ground motions are generated. These motions travel out from the source as body and surface waves (See fig C-2). As these waves travel farther out from the source, they are attenuated. The type and amount of attenuation depends on many factors, the most important of which are listed below:

—Size or source severity of the event on the source

—Type of fault mechanism

—Transmission path of the seismic waves from source to the site

—Vibration or wave frequency of interest of the seismic ground motion

—Distance from the source to the site

—Local site soil response effect

In estimating the type and severity of ground motion that would exist at a site due to some future seismic event, the analyst should incorporate the above parameters in his model. The current state-of-the-art methods for estimating the ground motion can be classified into two groups.

—Methods based on wave propagation theories through elastic and non-elastic media with appropriate damping characteristics.

—Empirical methods based on past data.

In the first method, various researchers in recent years have developed models to study displacement (or some other ground motion parameter) wave forms as a function of the type of event and the distance from the source. In particular, the models for estimating the surface wave patterns have been quite good and fit the data well (See Boore, (Biblio 12); Frazier, (Biblio 4); McCann, (Biblio 35)). There are some other models which look at the attenuation of Fourier spectra with distance. Such models take into account the damping characteristics of the transmission media, the wave frequency component of interest and the distance from the source. These types of developments are available for body waves (See Savy, (Biblio 53)). However, the most commonly used methods for ground motion estimation in engineering and for seismic hazard and risk analysis are the ones based on empirical relationships. In this manual, a short description of these empirical techniques will be presented. For a detailed study see Idriss (Biblio 30) or the OASES study by Woodward-Clyde Consultants (Biblio 70). It is commonly accepted by seismologists and geophysicists that the type, the amount, and the geometry of the rupture surface influences the amplitude and frequency of motion near the source. Other factors influencing the near-source motion characteristics are the velocity of rupture, the stress drop, the physical properties of the fault plane material, and the pattern of nonuniformity of rupture on the rupture surface. The larger the rupture surface, the greater the ground motion. However, there are definite upper limits for both the rupture size and the resulting motion. The wave patterns generated at the source travel out in all directions in the form of complex wave forms. The regions through which these wave forms travel from source to site constitute the "transmission path." It has been observed that the transmission path influences the attenuation of wave forms in both the frequency and amplitude domains. The decaying of amplitude with distance is usually referred to as the "attenuation." In the frequency domain, higher frequency components in the wave form get filtered out as the distance from the source to site increases. In this paragraph, only the amplitude attenuation will be discussed. Paragraph 3-6 considers the aspects of frequency attenuation and its influence on the response spectrum shape.

b. Empirical attenuation relations. Various empirical relationships are available in the literature to describe the relationship between the size of the event, the distance from the source and the site ground motion parameter of interest (see fig 3-21). In working with these relationships, the question of distance from the source to the site arises. The most "realistic" distance to be selected could be either the epicentral distance, hypocentral distance, distance from the site to the energy release center, or the distance from site to the closest rupture location on the fault. Earlier relationships have used epicentral distance; however, with the availability of more data in recent years, it has become evident that this distance is not the most relevant. Some studies have used hypocentral dis-

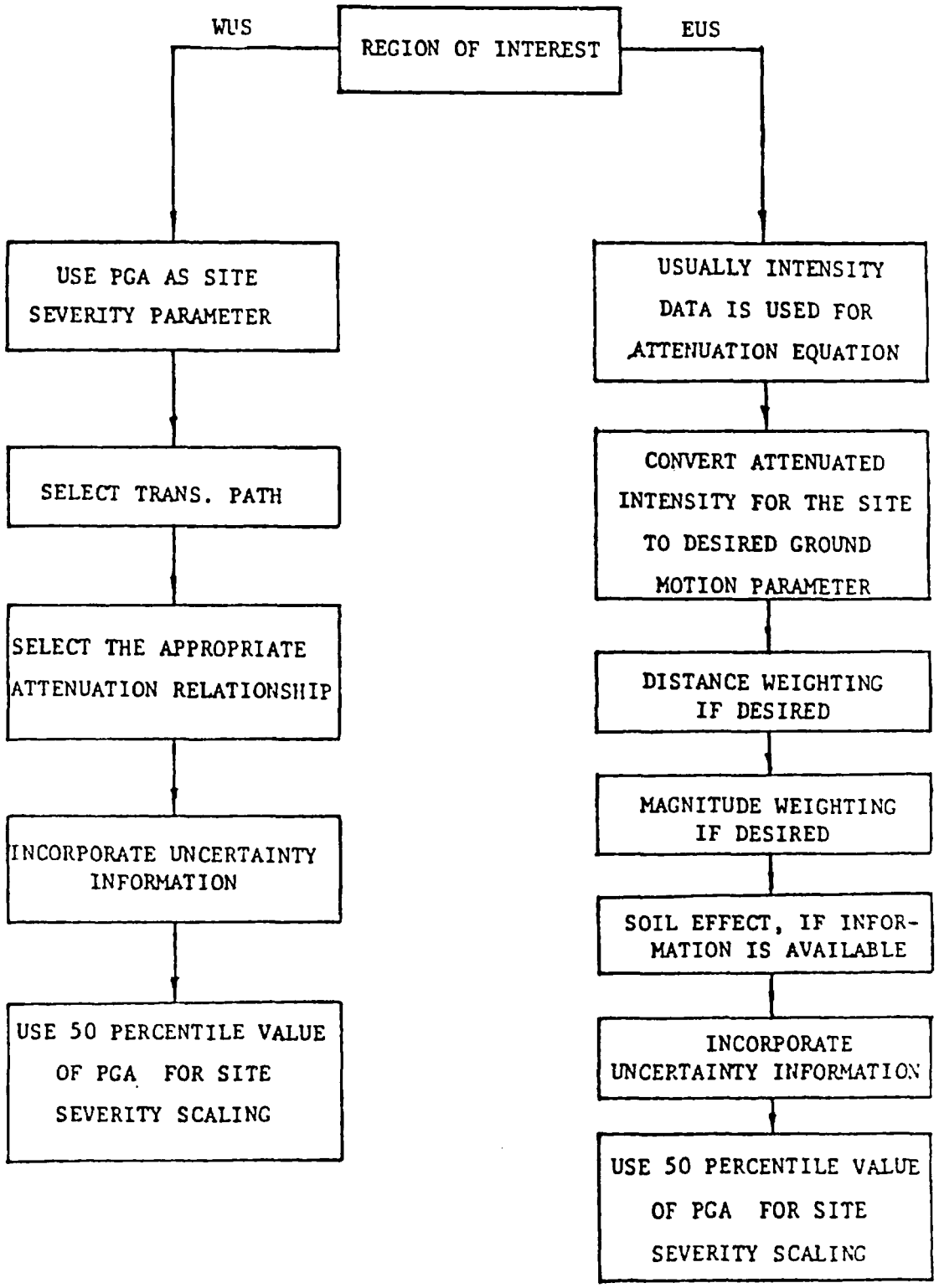

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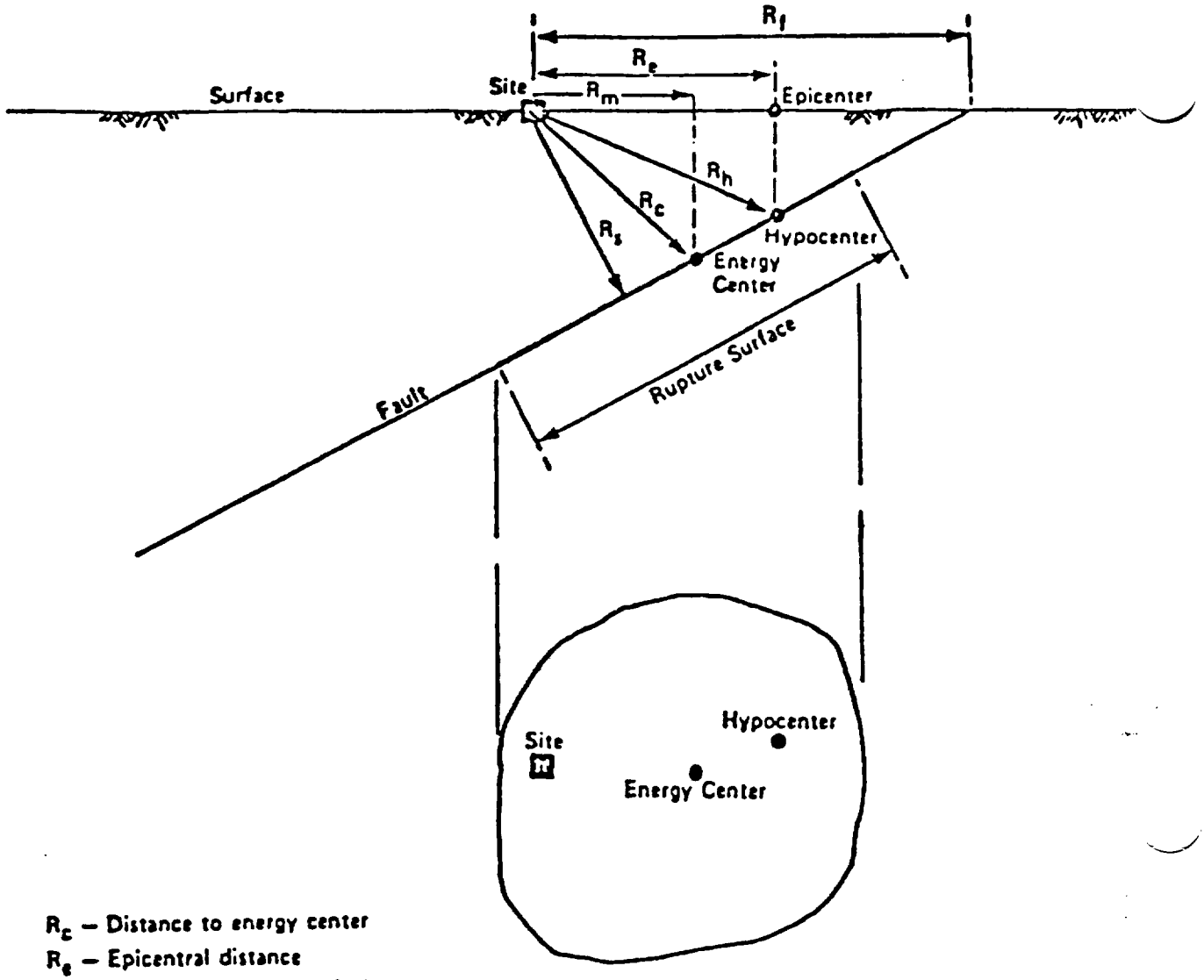
Figure 3-21. Step IV, attenuation of ground motion from source to site.

tance. The recent relationships use the concept of significant distance. This is the shortest distance to the ruptured source. Figure 3-22 shows some of the distance definitions used in the literature.



R_c – Distance to energy center
R_e – Epicentral distance
R_f – Distance to causative fault
R_h – Hypocentral distance
R_m – Map distance to energy center
R_s – Significant distance

Reprinted from "Offshore Alaska Seismic Exposure Study (OASES)," 1978, with permission from Woodward-Clyde Consultants.

Figure 3-22. Attenuation distances.

(1) Recent studies have indicated (OASES, Biblio (70) that the transmission path B is very important. Thus, for shallow earthquakes (transmission path A in fig 3-23) there is one attenuation relationship; whereas for deeper earthquakes, (transmission path B in fig 3-23) there is a separate attenuation relationship. This transmission path dependence has been observed in data collected in Alaska, Japan and in the Western United States. For most earthquakes in California and Hawaii, transmission path A should be assumed. Also, there are important differences in rates of attenuation for the WUS and EUS regions. These will be discussed in the paragraphs for these regions.

(2) Many empirical attenuation relationships are available in the literature. They all have their shortcomings in both accuracy and

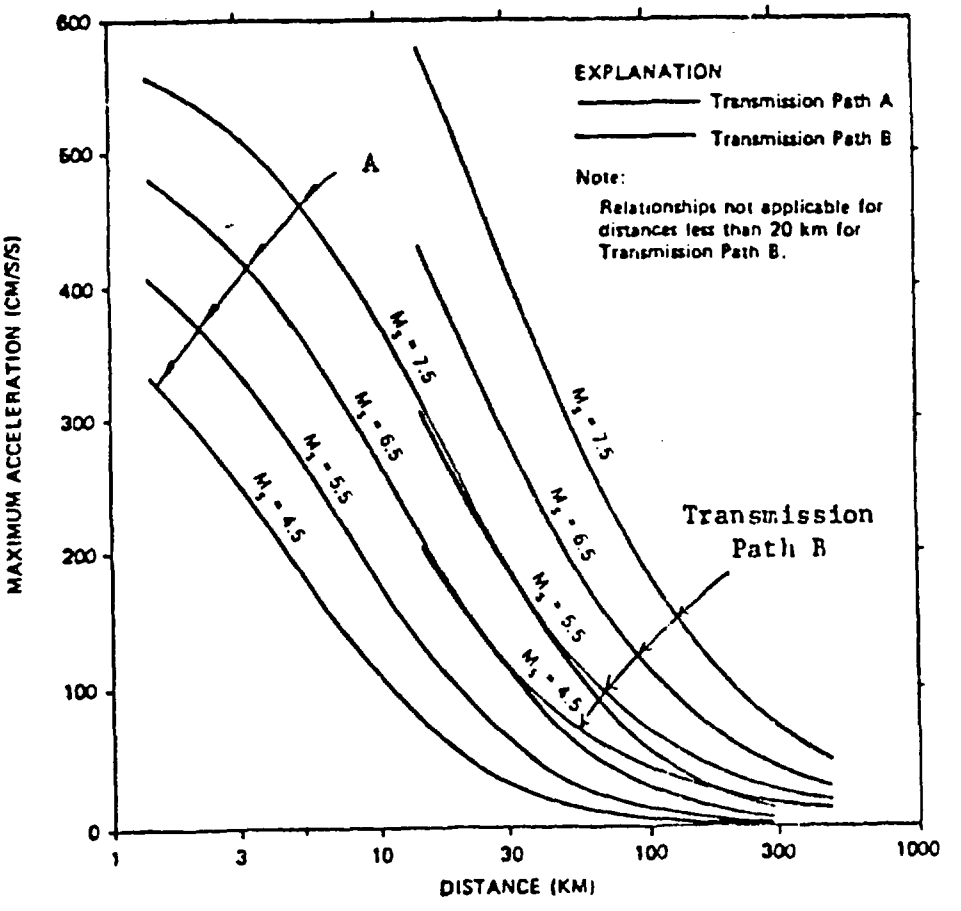


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Figure 3-23. OASES attenuation.

applicability for a given site. The scatter of data with respect to the estimated relationships is considerable. Hence, this scatter should be properly accounted for in the use of the attenuation relationships. See appendix D for an example.

c. Attenuation of ground motion in the Western United States. The abundance of strong motion records in the WUS makes empirical regression analysis the ideal tool to predict ground motion. A number of assumptions can have a significant impact on the results of such regression analyses. The most important ones are the attenuation mathematical forms, the regression techniques (linear, non-linear, weighted vs. non-weighted), the data base selection criteria, the definition of magnitude, attenuation, and site soil condition. Three of the most recent attenuation models developed for the WUS are given below:

—Campbell Model (Biblio 14)

—Joyner and Boore Model (Biblio 31)

—OASES Model (Biblio 70)

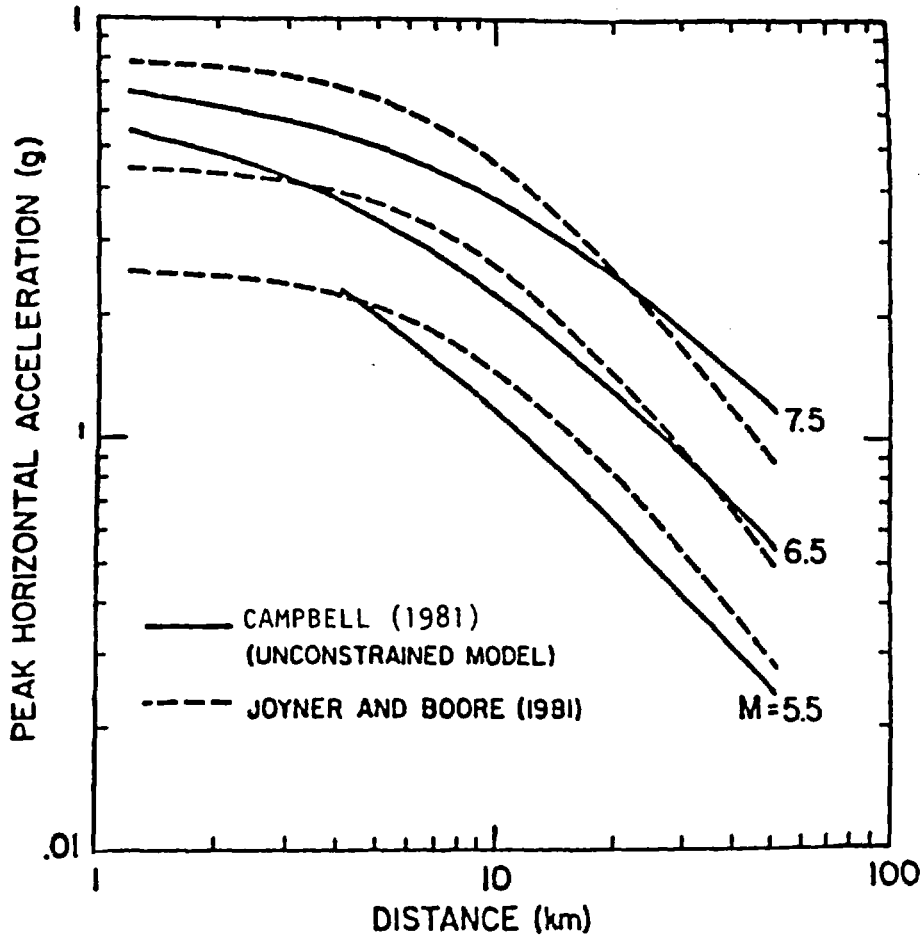
Figure 3-24 shows the first two of these relationships. The third relationship is given in figure 3-23.

(1) The mathematical relationship used for modeling the attenuation of peak acceleration with distance is expressed by Campbell (Biblio 14) by the equation:

$$PGA = a \exp(bM)(R + C(M))^{-d} \exp(-rR) \quad \text{(eq 3-16)}$$

where PGA is the mean of the peak acceleration scaled from the horizontal component of the accelerogram in g units.

M is the magnitude ($M = M_L$ for magnitude less than 6.0) ($M = M_S$ for magnitude greater than 6.0)



Reprinted from "Near-Source Attenuation of Peak Horizontal Acceleration," Campbell, K. W., and "Peak Horizontal Acceleration and Velocity from Strong Motion Records," Joyner, W. B. and Boore, D. M., Bulletin of the Seismological Society of America, Vol. 71, No. 6, 1981, with permission from the Seismological Society of America.

Figure 3-24. Attenuation Relations.

r is the absorption coefficient which affects the rate of attenuation.

R is the closest distance in kilometers to the surface projection of the rupture zone.

a, b, and d are regression constants. C(M) is a function which models possible nonlinear magnitude and distance scaling effects in the near field that may be supported by the data. According to Campbell,

$$C(M) = 0.567 \exp(0.345M)$$

Substituting this into equation 3-16 along with the values for a, b, and d gives the following equation for the median value of peak acceleration:

$$PGA = 0.22 \exp(0.734M)(R + 0.567\exp(0.345M))^{-0.891} \exp(-rR) \quad \text{(eq 3-17)}$$

where the value of r for the WUS is given by

$$r = 0.0423 - 0.00911M + 0.000573m^2 \quad \text{(eq 3-18)}$$

The 84th percentile value is obtained by multiplying equation 3-17 by 1.49. This step assumes that the natural logarithm of PGA has a standard error of 0.40.

The Joyner and Boore relationships (1981) are as follows:

$$\log A = -1.02 + 0.249M_o - \log R_1 - 0.00255R_1 + 0.26P \quad \text{(eq 3-19)}$$

$$\text{where } R_1 = (d^2 + 7.3^2)^{1/2} \quad (5.0 \leq M_o \leq 7.7) \quad \text{(eq 3-20)}$$

A is the peak horizontal acceleration in g units

M_o is the moment magnitude.

d is the closest distance to the surface projection of the fault rupture in kilometers.

P is zero for 50 percentile value and one for the 84 percentile value.

(2) The OASES (Biblio 70) relationship has the following mathematical format:

$$PGA = b_1 \exp(b_2 M)(R + C)^{b}3 \qquad \text{(eq 3-21)}$$

where PGA is the peak horizontal acceleration in cm/sec^2.

b_1, b_2, and b_3 are regression constants

R is the closest distance to fault rupture in kilometers.

C is a constant dependent on magnitude M, but independent of transmission path.

$$C = 0.864\exp(0.463M_s) \qquad \text{(eq 3-22)}$$

For different transmission paths and soil conditions, values of regression constants b_1, b_2 and b_3 along with the standard deviation of ln(PGA) are given in table 3-2. Use of any one of the three attenuation relationships should give reasonable results.

d. Attenuation of ground motion in the Eastern United States. Developing a ground motion model for the EUS is a difficult task for several reasons. First, there is not much strong motion data available from EUS earthquakes. Second, it is generally agreed that one cannot directly use a ground motion model developed for the Western United States (WUS) because data from a number of sources, e.g., Nuttli (Biblio 48), Chung and Bernreuter (Biblio 15) that the attenuation of seismic energy in the EUS is much different (more gradual) than in the WUS. Four approaches appear applicable to develop an EUS ground motion model. Given the limited amount of intensity data available for the EUS, three of the approaches use intensity as an intermediary variable to compare the ground motion between WUS and EUS:

Let I_s = site intensity

I_o = epicentral intensity

R = distance from source to the site

M = magnitude

F() and g() functional forms

GM = ground motion parameter, such as peak acceleration or peak velocity

Distance Weighting

$$I_s = f(I_o, R) \qquad \text{(EUS Data)}$$

Log GM = $g(I_s,R)$ and in some cases $G(I_s,M_L,R)$ (WUS D

Table 3-2. OASES attenuation constants for median PGA values.

		b_1	b_2	b_3	Standard Deviation of log (PGA)	Range of Magnitudes M_s
Path A (Shallow Focus Events)	stiff site	191	0.823	-1.56	0.568	4 to 7.5
	rock site	157	1.04	-1.90	0.579	4 to 7.5
Path B (Deep Focus or Subduction Zone Events)	stiff site	284	0.587	-1.05	0.70	5 to 8.5
	rock site	276	0.68	-1.20	0.70	4 to 8.5

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Magnitude weighting

$I_s = f(I_o,R)$ (EUS Data)

$\text{Log GM} = g(I_s,M)$ (WUS Data)

No weighting

$I_s = f(I_o,R)$ (EUS Data)

$\text{Log GM} = g(I_s)$ (WUS Data)

The fourth method uses a theoretical approach such as Nuttli's (Biblio 48) model. It combines theoretical modeling with measured regional Q values (damping value of the transmission medium), assumes the near-source ground motion in the EUS is the same as in the WUS, and scales only by magnitude. If it is kept in mind that the elements of ground motion models are a combination of source travel path and local site effects, it can be seen that all four approaches make a common assumption. This is that the set of WUS earthquakes, making up the strong ground motion data set, adequately represents future earthquakes in the EUS in terms of such parameters as dynamic stress drop, static stress drop, seismic moment, and focal mechanism. Validity of this common assumption can be verified only as more information is generated in the future.

(1) *Empirical models using an intensity attenuation data base.* The first three approaches require a relation giving the attenuation of intensity as a function of distance. It would be ideal to have a number of earthquakes with a range of epicentral intensity (I_o) and many reports of site intensity (I_s) for each earthquake. Then it would be possible to obtain the required relation of the form through a simple regression analysis:

$$(I_s - I_o) = C_1 + C_2R + C_3\ln R \qquad \text{(eq 3-23)}$$

However, no such data set exists in a usable form. Considerable data does exist, but it is in the form of isoseismals for given earthquakes. Isoseismals have a number of drawbacks, including the fact that they are generally subjectively determined. Of even greater significance is the fact that isoseismals represent the average distance at which a given intensity was felt, rather than average intensity at a given distance. Six earthquakes, that have been studied in enough detail to develop sufficient data for determining the required coefficients in equation 3-23 by regression analysis, are listed below.

Name	*Date*	*Maximum Intensity*	*Analysis Source*
Southern Illinois	11-9-1968	VII	G.A. Bollinger (Biblio 9)
Cornwall-Massena	9-4-1944	VII	R.J. Holt (Biblio 9)
Ossippee	12-20-1940	VII	R.J. Holt (Biblio 9)
Giles County	5-31-1897	VII-VIII	G.A. Bollinger (Biblio 9)
Charleston	8-31-1886	X	G.A. Bollinger (Biblio 9)
New Madrid	1811-1812	XI-XII	O. Nuttli (Biblio 9)

(2) *Strong ground motion data base.* This data base allows correlation of site intensity with such information as peak ground acceleration (PGA), velocity (PGV), distance from recording site to the epicenter and/or nearest approach of the fault rupture plane, earthquake magnitude, and information about site geology (See paragraph C-1, appendix C-1). A number of such data bases have been developed, e.g., Murphy and O'Brien (Biblio 43), Trifunac and Brady (Biblio 66), McGuire and Barnhard (Biblio 38), Boore et al., (Biblio 13). If the site intensity is to be correlated with spectral amplitude as well as PGA, the data sets are more limited. The most common one consists of the California Institute of Technology (CIT) data tapes, such as those of Trifunac and Brady or McGuire and Barnhard. These sets are then used to obtain relations of the form:

$$\ln GM = C_1 + C_2I_s + C_3 \ln R \qquad \text{(eq 3-24a)}$$

or

$$\ln GM = C_1 + C_2I_s + C_3M \qquad \text{(eq 3-24b)}$$

or

$$\ln GM = C_1 + C_2 I_s \quad \text{(eq 3-24c)}$$

and

$$\ln GM = C_1 + C_2 M + C_3 \ln R + C_4 S \quad \text{(eq 3-24d)}$$

where

GM = Ground motion parameter (PGA, PGV, or spectrum S_a at a given period)

I_s = Site intensity

R = distance measure (epicentral, closest approach, etc.)

M = generally local magnitude

S = Site type parameter (for soil S = 0; for rock S = 1)

The parameters C_i are determined by regression analysis using an appropriate data set. The values of I_s, R, and site type for some records differ significantly between data sets. Thus some choices are involved.

(3) Site Correction Factor. The ideal way to include a generic correction factor for rock sites is to perform the required regression analysis using only the rock subset of the data in place of equation (3-23) one could use:

$$I_s - I_o = C_1 + C_2 R + C_3 \ln R + C_4 S \quad \text{(eq 3-25)}$$

where S = site type (S = 0 for soil and S = 1 for rock), and in place of equations 3-24a, b, c, one could include a site type in the relation between ground motion, site intensity, and distance or magnitude. Unfortunately, the intensity attenuation data does not include the site type and the intensity assigned is not generally at a site where an accelerograph would be located, but rather it is determined from isoseismals or nearby reports of intensity. This reduces the applicability of the above approach.

(a) Another method consists of introducing the variation between soil and rock sites at the level of equations (3-24) and the general ground motion model for the EUS is the combination of equation (3-23), the appropriate form of equations (3-24) and the inclusion of the term, C_4S (S = 0 for soil sites and S = 1 for rock sites) where C_4 is obtained from WUS data, ($\ln(GM = C_4S + C_1 + C_2M_L + C_3\ln R)$. The resulting ground motion is of the form:

$$\ln GM = C_1 + C_2 I_o + C_3 R + C_4 \ln R \quad \text{(eq 3-26)}$$

where GM is PGA, PGV or any spectral ordinate S_a of interest. Several models are plotted for PGA in figure 3-25. Based on the methods suggested in this section, any one of the following four attenuation relationships can be used.

1. Gupta and Nuttli model (1976). (Biblio 25).
2. Bollinger model (1977). (Biblio 8).
3. Ossippee model (1977). (Biblio 64)
4. Model developed by Tera Corporation.

(b) The Tera Model is based on the first three models mentioned above. This model has the following format:

$$\log PGA = 0.74 + 1.12\, m_b - 0.733 \ln R - 0.0007R \quad \text{(for } R > 20 \text{ kilometers.)}$$

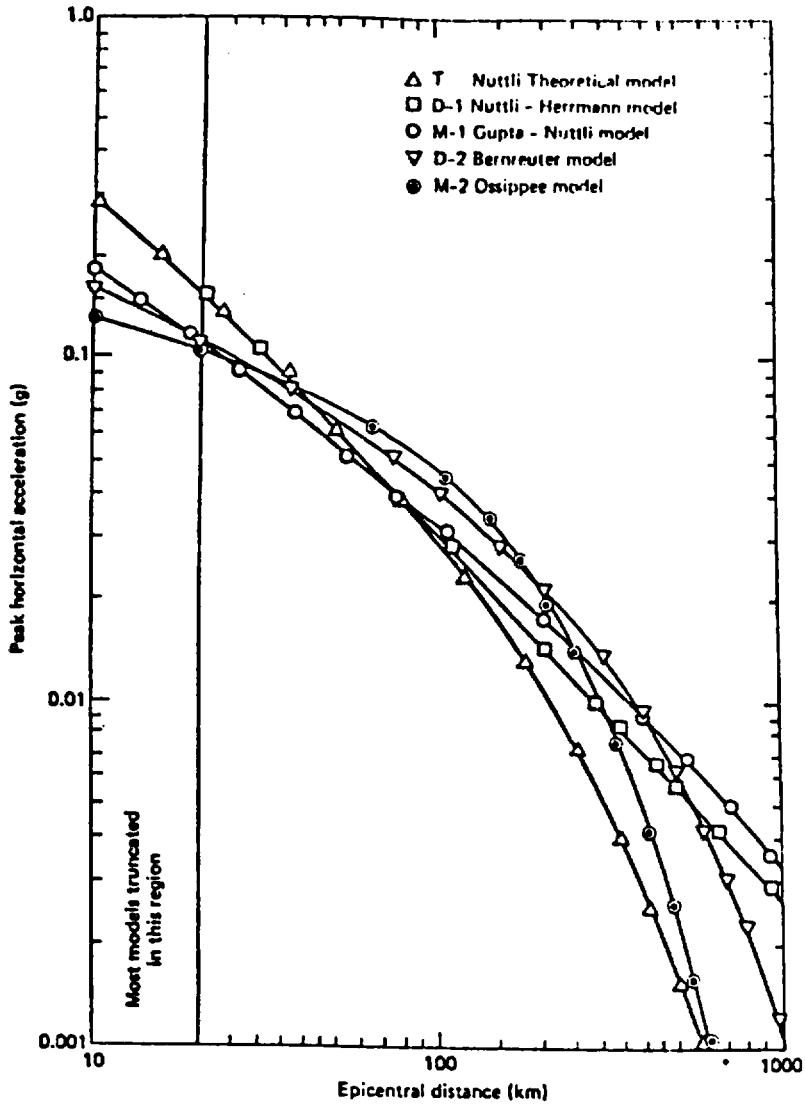
$$= -1.47 + 1.12\, m_b \quad \text{(for } R \leq 20 \text{ kilometers.)}$$

PGA is in cm/sec^2

M_b is the body wave magnitude = $(0.98M_L - 0.29)$

R is the epicentral distance in Kms.

e. Uncertainty associated with ground motion model applied in the east. One weakness of the approach applied in the EUS has to do with apportioning an attenuation model into submodels. The uncertainty contained in each of the submodels increases the uncertainty in the final prediction (Cornell, et al., (Biblio 20). Although at the present time, there does not appear to be any rational alternative to this. This added uncertainty significantly influences the seismic hazard results. Improved estimates could be obtained through additional work on this topic. When an attenuation model is derived *directly* from recorded ground motion, the statistical uncertainty usually corresponds to a one standard deviation confidence level of 1.6-2.0 times the mean. When the uncertainty in mean predictions of intermediate parameters (such as intensity) is rigorously included, this multiplicative factor becomes 2.0-2.9 (Cornell, et al., (Biblio 20). A hazard analysis, which results in a one standard deviation confidence level equal to 2 or 3 times the mean predicted value of site severity is being dominated by this multiplicative factor. It should be recognized that a large part of the uncertainty is due to the use of data representing all possible earthquake types and all possible travel paths. The necessity for this is to acquire a sufficient statistical sample size for averages and empirical prediction equations. However, in most cases the seismic hazard at a particular site is largely determined by a



"Seismic Hazard Analysis-Solicitation of Expert Opinion," Nuclear Regulatory Commission, reprinted from NUREG/CR-1582, Vol. 3, 1980.

Figure 3-25. Comparison of ground motion models for M_b = 5.5.

particular type of earthquake (e.g., magnitude range, depth, focal mechanism, etc.), with a particular path. It is believed that a detailed consideration of this specific local knowledge would significantly reduce the attenuation model uncertainty. Also, as stated in the next paragraph, the median forecasted value of PGA is used for scaling the response spectrum shape. The high uncertainty in actual PGA values does not enter into this scaling procedure; only the statistical sampling uncertainty of predicted median PGA as it estimates the true (infinite sample size median value) median is of concern. Aside from the use of sub-models (such as conversion of I to M), there is no *a priori* reason to believe that the random uncertainty associated with prediction of median PGA levels in the EUS should be substantially different than in the WUS for given parameters. Therefore uncertainty measures similar to those values obtained in the WUS from direct regression on strong motion data are recommended for use in the EUS.

f. Site severity for scaling the response spectrum shape. For the purpose of scaling the appropriate site response spectrum shape (DAF) as described in the next paragraph 3-6, it is recommended that the median or 50 percentile value of PGA be used in the attenuation equation. The mean value shall be used if the median is not given by the attenuation equation. For a given

convoluted seismic hazard or return period of severity at the site, it is judged that the median value is sufficiently conservative for spectral scaling purposes. Note that PGA data used for empirical attenuation relations is the PGA from the principal component of the recorded time history. Further conservatism due to the spectral enveloping property of the specified DAF shape is discussed in paragraph 3-7.

g. Computation of total hazard at the site. The process of computing the hazard or probability of exceeding a given level of site intensity (such as PGA) involves the convolution of the probabilities of all the possible combinations of source intensities (M or I) and attenuation distances R that can produce or exceed the given level of PGA. Figure 3-26 provides a simplified illustration of the typical condition for a line source and an area source.

(1) On the line source the set of all possible combinations of rupture length location, its corresponding attenuation distance R_i and mag-

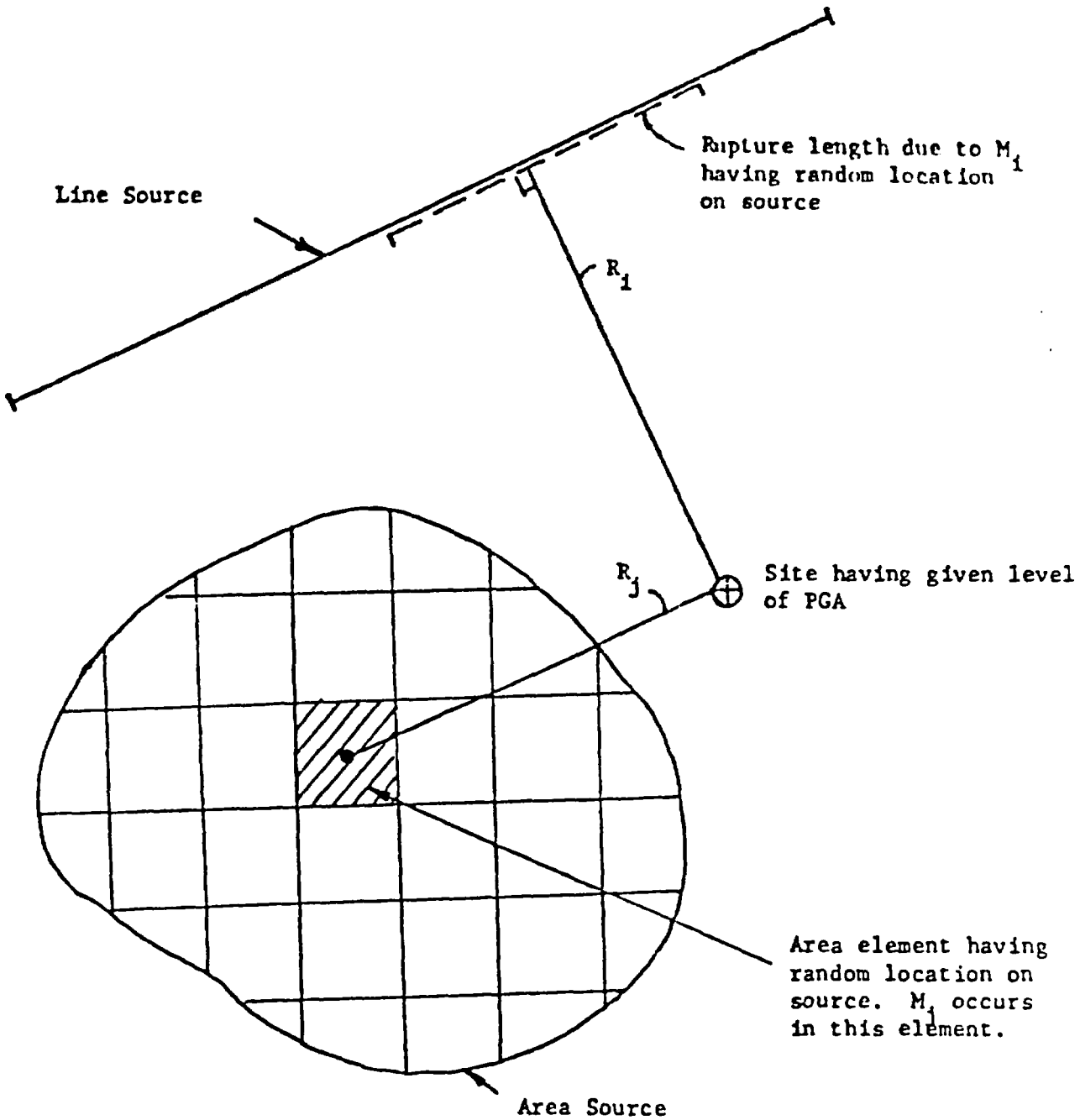


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Figure 3-26. Description of sets of M and R required for a given PGA.

nitude M_i are able to produce or exceed the given PGA at the site. Similarly the set of the M_j and area element location R_j produces the PGA at the site from the area source.

(2) The total probability of exceeding the PGA is the probability of the union of the occurrences of all the sets of M_i and R_i combinations on the line source, and M_j and R_j combinations on the area source. The convolution operation required for this total probability for a selected range of given PGA values can be very lengthy and is best performed by a computer program such as STASHA (Stanford University Technical Report, No. 36). A simple example of this type of calculation is given in paragraph 3-7*c*.

(3) Finally, a sensitivity analysis involving the probable upper and lower bound values of the parameters of the hazard analysis may be performed. For example, when large uncertainties exist due to sparse data and (or) judgementally assigned values in source locations R, recurrence parameters α, β, M_{max}, and different but applicable attenuation relations, then separate runs of PGA evaluations may be performed using probable upper and lower bounds for each individual parameter. The results of this analysis are useful to identify the important factors that significantly effect the calculated PGA, such that perhaps more information can be obtained to better evaluate these factors of parameters. Also, the resulting probable bounds on a PGA for a given return period provide a numerical description of the quality or stability of the hazard analysis and can assist in the final assignment of the design spectral scaling value for the PGA.

3-6. Site specific response spectra, step V.

The exact prediction of future ground motions (such as the accelerogram x(t)) at a site is not possible. Therefore, forecasted response spectra representative of this motion offer the most effective method of specifying the future. Having the value of site severity from step IV of the seismic hazard analysis, this value provides the basis for scaling the response spectrum shape resulting from step V, treated in this paragraph, and summarized in figure 3-27.

In practice, the response spectrum shape may be obtained by three rather common techniques; two of which are empirical, and one analytical method:

—Averages of Normalized Spectra

—Attenuation of Spectral Ordinates

—Analytical Soil-Column Response

(see para C-3, app C for an overview of all methods). The results of any or all of these methods may be combined to define the appropriate spectra for structural design and analysis; this is usually done in a rather subjective manner to best represent the quality of information from each method, (See SEAOC Pamphlet, (Biblio 55)). However, before proceeding to the description of these methods and the formation of the site specific spectra, it is useful to review the major factors that govern the shape and size of the response spectrum.

a. Spectral shape factors. It is generally recognized that the frequency content and corresponding response spectrum shape is governed by the following source and site factors.

—Characteristics of Soil Deposits Underlying the Site

—Magnitude of Seismic Event producing the Site Ground Motion

—The Source Fault Rupture Characteristics

—The Source-to-Site Travel Path Characteristics of Distance and Wave Attenuation Properties

The second and third factors are recognized subjects of research, but are not generally incorporated in site spectra with the exception that records for spectral averaging purposes may be grouped according to magnitude levels. The first "soil type" factor is well established and used in most site specific ground motion studies. The fourth "travel path" factor is also an established procedure for both distant sites in all regions, and for the representation of the low attenuation rates in the Eastern United States. Detailed discussions and procedures for determination of spectra are given in appendix C, paragraphs C-2 and C-3.

b. Statistical averages of normalized response spectra. In this first empirical method, the shape of the spectrum is determined by a statistical analysis (evaluation of averages and standard deviations) of past earthquake strong motion accelerograms; as classified according to site conditions, distance from the source and size of the event. All the response spectra for a common set of conditions are normalized by the recorded PGA, see figure 3-28.

The mean and standard deviations of the normalized spectra (referred to as the Dynamic Amplification Factor or DAF) are then calculated. This statistical summary is used to fore-

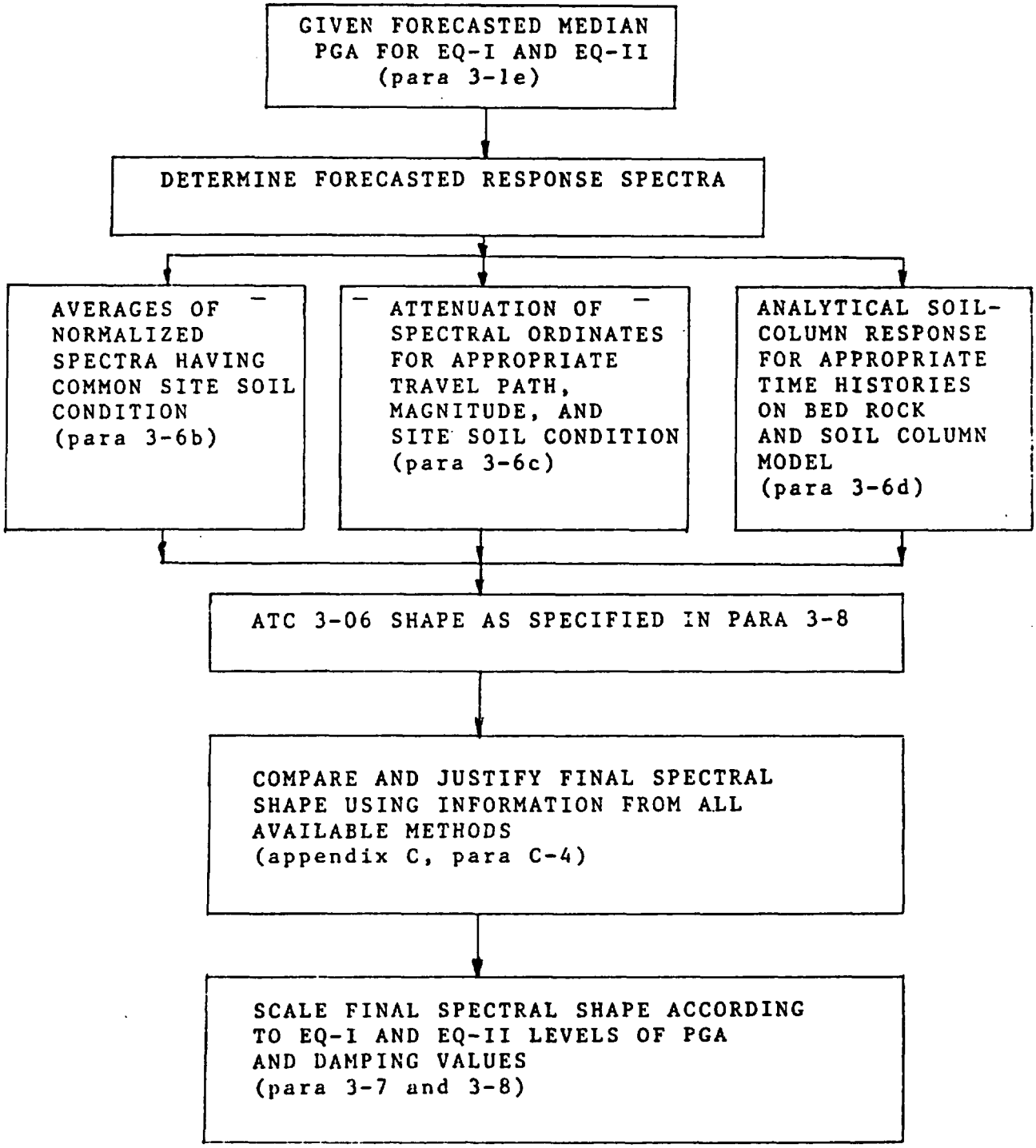

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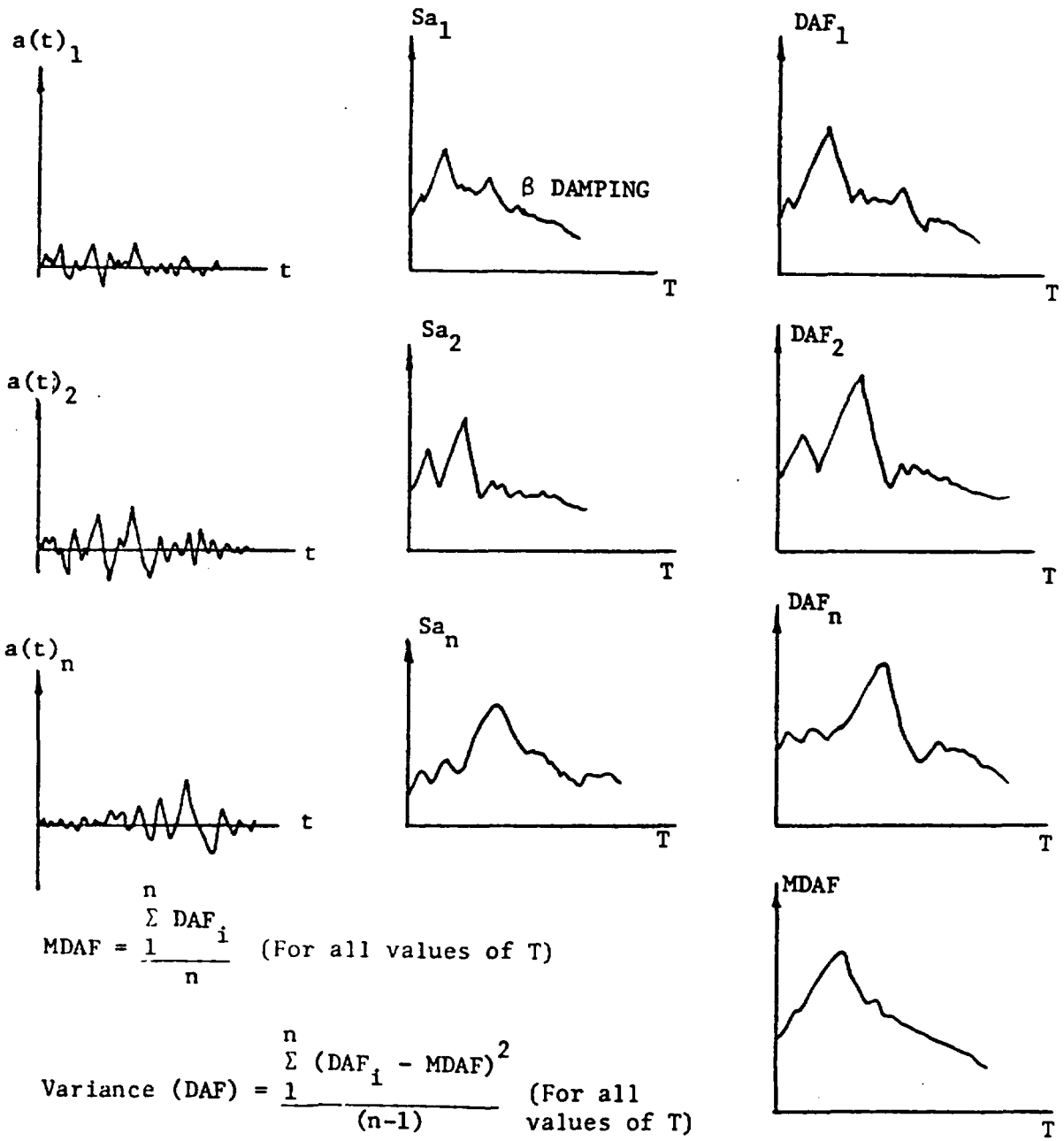
Figure 3-27. Step V, site specific response spectra.



$$\text{MDAF} = \frac{\sum_{1}^{n} \text{DAF}_i}{n} \quad \text{(For all values of T)}$$

$$\text{Variance (DAF)} = \frac{\sum_{1}^{n} (\text{DAF}_i - \text{MDAF})^2}{(n-1)} \quad \text{(For all values of T)}$$

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Figure 3-28. Statistical averaging of normalized spectra.

cast the spectral shape of future events according to the particular site conditions. The method, even though widely used for practical applications, has some shortcomings. The procedure of normalizing according to PGA creates a large coefficient of variation (standard deviation divided by average), particularly in the long period region. However, since no better means of normalization is yet available, this technique has provided the primary source of design earthquake spectral shapes. See Seed et al. (Biblio 56), Kiremedjian and Shah (Biblio 33) and ATC 3-06 (National Bureau of Standards, Special Publication 510).

c. Attenuation of spectral ordinates. The second empirical approach of forming a site spectrum is by the use of attenuation equations for spectral ordinates at specific period values for a set of records and then statistically analyzing these attenuated ordinates. This again provides a mean and standard deviation description of the site spectrum such that an upper confidence limit can be given in terms of one or more standard deviations. This method has the advantage of avoiding a normalization method with its inherent creation of large spectral variability. This advantage is offset, however, by the need for the use of spectral attenuation relations that have large prediction error. Also, the development of these relations requires a sufficient set of records applicable for a common seismic region; the method is therefore limited to these regions (see app C, para C-3*d*). This method, however, may find increased applicability in the Eastern United States (see Nuttli: Biblio 47), not because of the availability of data for that region, but because the method can incorporate expert opinion and theories for wave transmission peculiar to the region and its postulated sources of seismicity. The most current application of this technique is given by NUREG/CR—1582, Vol. 3 and 4, (Biblio 63, 64).

d. Analytical soil column response. The third or analytical method of obtaining a spectral shape is based on a site specific study of the strong motion accelerogram. If the acceleration time history at the bedrock level for a given site can be formulated, then using the overlying soil layers as a filter, the response on the surface can be determined. Thus, the transfer function of the soil layer and the motion at the bedrock level determines the time history and corresponding spectral shape at the surface. The problem with this method is that a time history at the bedrock level has to be formulated. This may not be an easy task for a region where the seismotectonic information is not complete. See appendix C, paragraph C-3.

e. Site specific earthquake spectra. The procedures of paragraphs 3-6*b*, *c*, and *d*, have all or in part lead to generalized versions of earthquake spectra. Some of the important recommendations resulting from these procedures are given here and in the next paragraphs on shape effects. These include the methods of:

—Newmark-Hall, (Biblio 44)

—Seed et al, (Biblio 56)

—Kiremidjian and Shah, (Biblio 33)

—ATC 3-06

(1) Newmark-Hall Method of Constructing Elastic Response Spectrum. This is an empirical method of constructing an elastic spectrum. It employs the following normalized values for ground motion:

Acceleration	1g
Velocity	48 in/sec.
Displacement	36"

Thus, for a peak ground acceleration of interest, as forecasted for the site, construct the ground motion parameters on the tripartite plot. As an example, let the PGA value be 0.35g. For this case, ground motion values are:

Acceleration A	= 0.35g (1g × 0.35)
Velocity V	= 16.8 in/sec. (48 in/sec × 0.35)
Displacement D	= 12.6" (36" × 0.35)

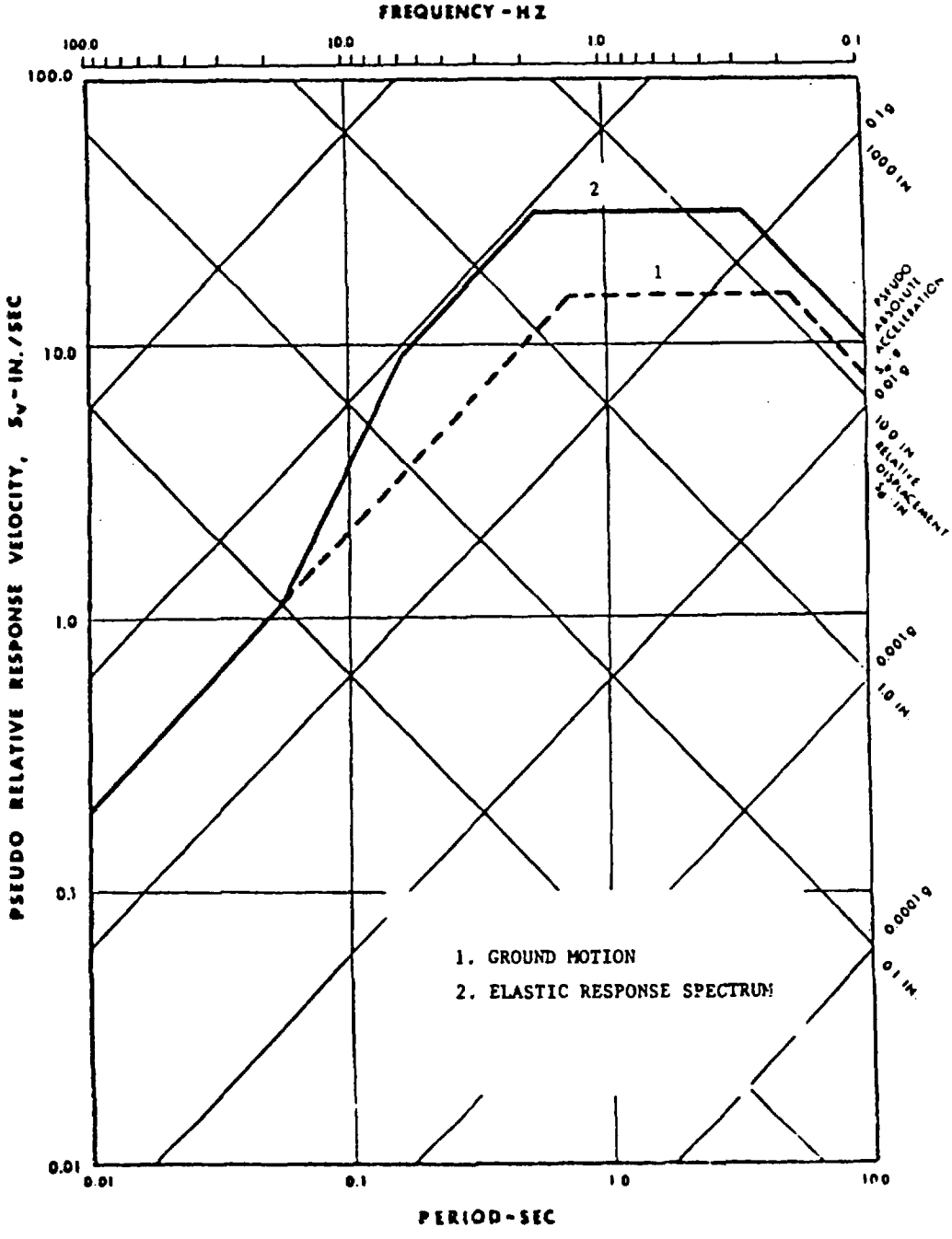
Draw this ground motion spectrum on the tripartite paper. (fig 3-29).

(a) The second step is to construct an "elastic" response spectrum. To construct this spectrum, a table of amplification factors, based on the study of past spectra, is available. See table 3-3 from (Biblio 44).

These amplification factors are functions of damping ratios, and the described confidence level. As an example, consider a 5 percent damping ratio, and the median level.

(b) The lines of constant acceleration, velocity, and displacement representing the elastic response spectrum are given by the corresponding ground motion values times the appropriate factors from the table.

$$S_a = (.35g)(2.12) = 0.74g$$



Reprinted from "Earthquake Spectra and Design," Newmark, N. M. and Hall, W. J., EERI Monograph Series, 1982, with permission from the Earthquake Engineering Research Institute.

Figure 3-29. Newmark-Hall Spectrum.

Table 3-3. Spectrum amplification factors for horizontal elastic response.

Damping,	One Sigma (84.1%)			Median (50%)		
% Critical	A	V	D	A	V	D
0.5	5.10	3.84	3.04	3.68	2.59	2.01
1	4.38	3.38	2.73	3.21	2.31	1.82
2	3.66	2.92	2.42	2.74	2.03	1.63
3	3.24	2.64	2.24	2.46	1.86	1.52
5	2.71	2.30	2.01	2.12	1.65	1.39
7	2.36	2.08	1.85	1.89	1.51	1.29
10	1.99	1.84	1.69	1.64	1.37	1.20
20	1.26	1.37	1.38	1.17	1.08	1.01

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$S_v = (16.8 \text{ in/sec})(1.65) = 27.7 \text{ in/sec}$

$S_d = (12.6'')(1.39) = 17.5 \text{ in.}$

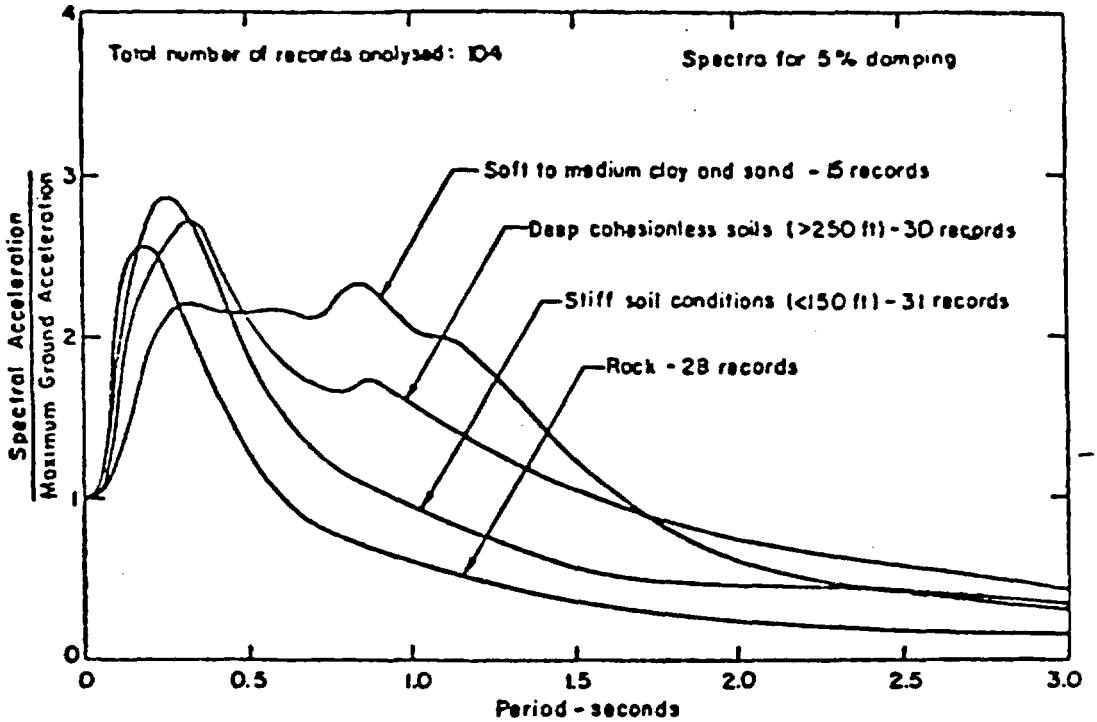
(c) These constant levels are plotted on the tri-partite paper, and along with recommended connecting lines as given in (Biblio 44), the complete spectrum is defined. This Newmark/Hall method provides a direct procedure of forming a spectrum, and also has the advantage of constructing inelastic yield force and deformation spectra in terms of structural ductility factors (see Biblio 44). Also the site soil conditions can be represented by either the known forecasted peak ground velocity or prescribed relations between peak ground acceleration, velocity, and displacement. However, since the representation and description of site soil conditions are not as detailed as in the following methods, the use of this Newmark/Hall method is not recommended except for general comparison with other methods.

(2) *Seed et al.* This method provides mean DAF, and mean plus one standard deviation shapes for different categories of site conditions, see figures 3-30 and 3-31. These DAF shapes may be scaled to the forecasted PGA value having a given risk value at the site.

(3) *Kiremidjian and Shah.* This method is similar to method (2), and a definite listing of the data base and the site soil conditions is provided. Also, in addition to mean and mean plus one standard deviation shapes, probability functions are given for the random DAF values as they are scattered about the mean value. This probability information is most useful for calculating the total risk of exceeding a specified response spectrum. This total risk must involve the convolution of probability functions for both the forecasted PGA scaling factor and the DAF spectral shape. See Kiremidjian and Shah (Biblio 33) for examples. A more simplified reliability calculation is given in paragraph 3-7.

(4) The ATC 3-06 method uses much of methods (1) and (2) as background justification. It, however, goes further to provide simplified DAF shapes for not only the soil types but also the tectonic region. Because of this simple, yet representative quality, it is recommended that these ATC 3-06 shapes be used for the appropriate site conditions and tectonic region. Therefore, unless there are special site conditions, close active sources, or high risk facilities, these shapes as scaled by the forecasted site severity values can provide the input spectra for design and analysis. The complete ATC 3-06 method for site severity and response spectra is given in paragraph 3-8. In order to represent the particular regional attenuation effects that are indicated when the A_v value exceeds the A_a value on the contour maps given in paragraph 3-8, the spectral shape should be found using the respective contour map values of A_v and A_a, then this shape should be scaled by the ratio of the forecasted PGA to the contour map value of A_a. The PGA value corresponds to the hazard level or return period of EQ-I or EQ-II.

f. Factors affecting response spectral shapes. As mentioned in paragraph 3-6*a* there are several important conditions or factors that can alter the shape or frequency content of the response spectrum.



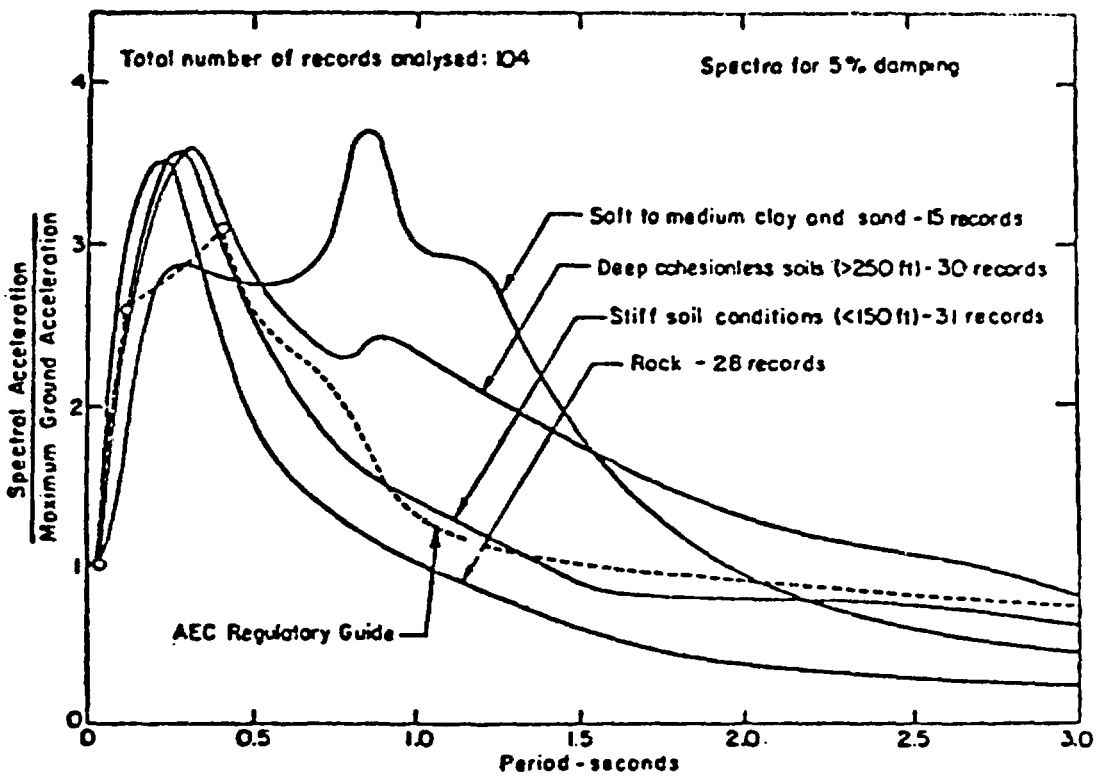
Reprinted from "Site-Dependent Spectra for Earthquake Resistant Design," Seed, H. B. et al, Report No. EERC 74-12, University of California at Berkeley, 1974.

Figure 3-30. Average acceleration spectra for different site conditions.

(1) *Type and duration of fault rupture.* Generally the type and duration of the fault rupture affects the frequency content of the seismic wave. Various seismological papers are available which describe the theoretical formulation of the above mentioned dependence. (Haskell, (Biblio 28, 29); Savage, (Biblio 51)). According to these models, the seismic wave characteristic in the time and frequency domain is a function of the radiation pattern (source and propagating geometry), seismic moment (size of the event or energy release level) and the source mechanism.

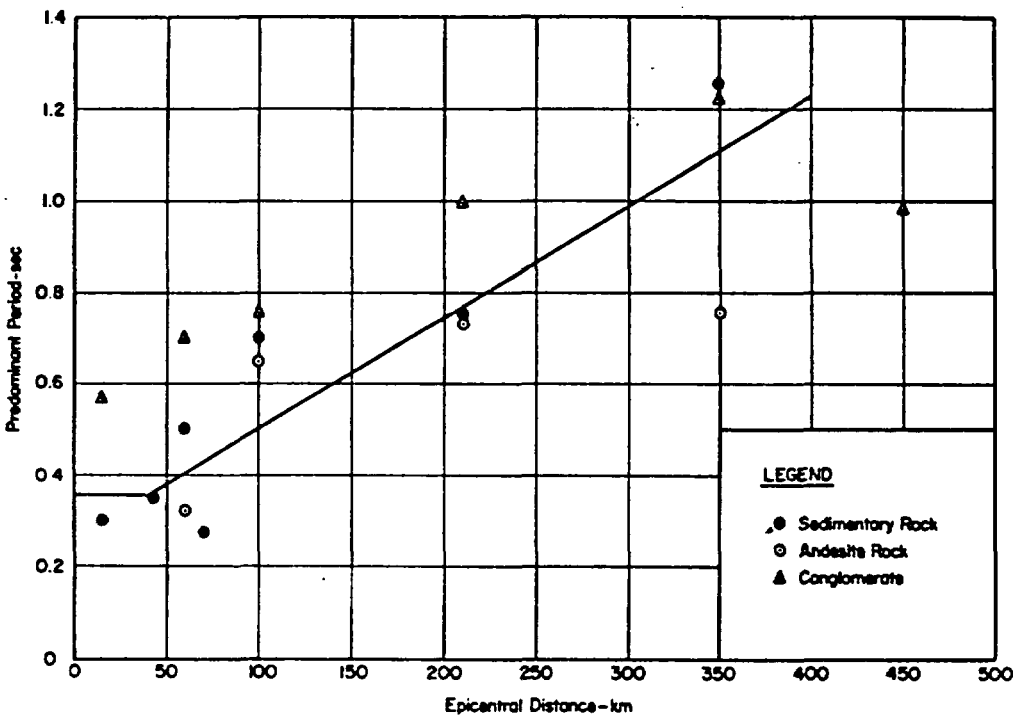
(2) *Size of event in terms of magnitude or seismic moment and distance from source to site.* Based on the recorded ground motion characteristics, many empirical relationships are available to show the dependence of the response spectrum shape on the size of an event, the distance from the source to the site and the predominant period (or frequency) of the motion. Figures 3-32 and 3-33 show such empirical results. It can be seen from these figures that the higher frequency components are filtered out from seismic waves as the distance from the source to the site increases. In other words, the predominant period of motion increases with distance and size of the seismic event. The engineering implication of this observation is obvious. Taller structures are affected more by large distant earthquakes than are the shorter (or stiffer) structures at the same location.

(3) *Local site soil conditions.* The effects of local site soil conditions on the frequency content can be very significant. The response of a given layered soil media to a seismic bedrock motion depends heavily on the transfer function of the soil. Thus, stiffer soils transfer higher frequency components whereas softer soils transfer lower frequency components. Extensive studies of the available strong motion accelerograms by many researchers have shown that the shape of the Response Spectrum changes with the site condition. There are usually three classifications of soils: soft alluvium deposits (soil class 0), intermediate stiff soils (soil class 1) and firm soils or rocks (soil class 2). These classifications could be made on the basis of shear wave velocities. As a guide to such a possible



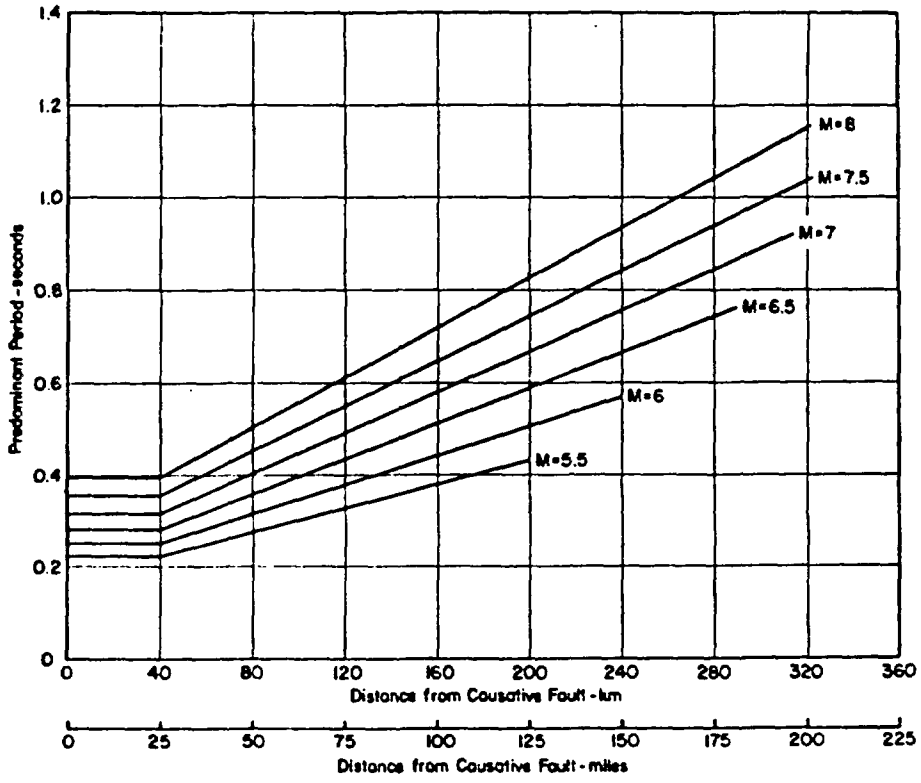
Reprinted from "Site-Dependent Spectra for Earthquake Resistant Design," Seed, H. B. et al, Report No. EERC 74-12, University of California at Berkeley, 1974.

Figure 3-31. 84 Percentile acceleration spectra for different site conditions.



Reprinted from "Characteristics of Rock Motions During Earthquakes," Seed, H. B. et al, Report EERC 68-5, University of California at Berkeley, 1968.

Figure 3-32. Predominant periods for motions in rock-earthquake magnitude = 7.



Reprinted from "Characteristics of Rock Motions During Earthquakes," Seed, H. B. et al, Report EERC 68-5, University of California at Berkeley, 1968.

Figure 3-33. Predominant periods for maximum accelerations in rock.

classification, the following procedure is recommended (V_S is the shear wave velocity):

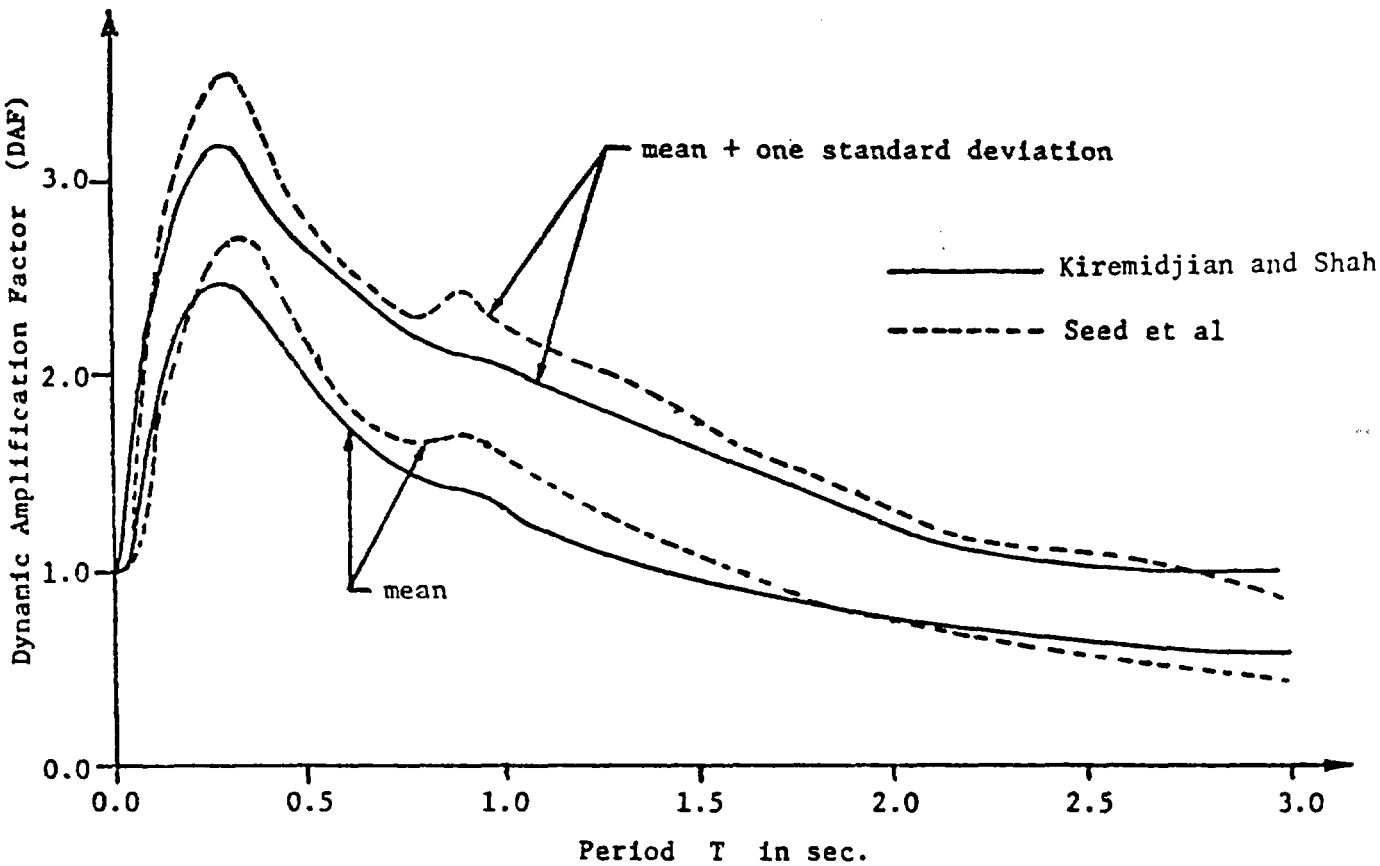
Firm Site: $V_S \geq 450$ meters/sec.

Intermediate Stiff: $250 \leq V_S < 450$ meters/sec.

Soft Alluvium Deposits: $V_S < 250$ meters/sec.

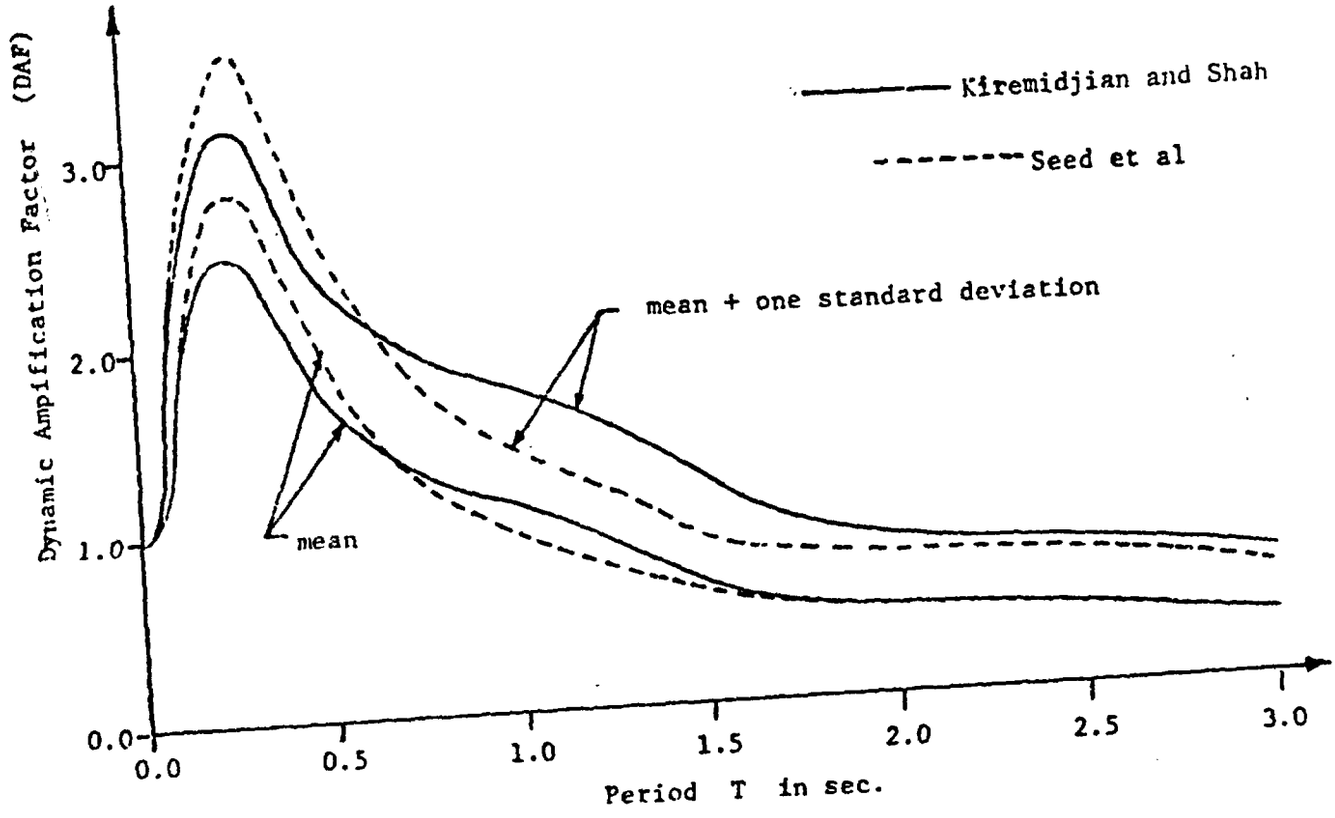
Also, for the purpose of this manual, these soil classes 0,1,2 may be considered to correspond to the soil types S_1, S_2, S_3 respectively, as described in table 3-5. Figures 3-34, 3-35 and 3-36 taken from Kiremidjian and Shah, (Biblio 33), show the effect of the soil conditions on the frequency content of ground motion. It can be seen from these figures that for soil class 0, the spectral peak occurs at higher period than for stiffer soils of class 2. Under very special conditions, (such as in Mexico City, where the city is on an old lake bed), the spectral peak could occur at a period as long as 1.5 to 2.5 seconds.

(4) *Regional geology.* This is a most important effect, not only for the Western United States where there is a reasonable amount of strong motion records, but for the Eastern United States where data is sparse and predictions of future ground motion must be based upon geological features. The future developments in ground motion prediction will depend strongly upon inferred behavior of possible earthquake source mechanisms, and the corresponding propagation of effects in the general geological structure. One of the most prominent characteristics of Eastern United States seismicity is the exceptional transmission of peak



Reprinted from "Probabilistic Site-Dependent Response Spectra," Kiremidjian, A. S. and Shah, H. C., Journal of the Structural Division, Proceedings of the ASCE, Vol. 106, No. ST1, January 1980, with permission from the American Society of Civil Engineers.

Figure 3-34. Comparison of DAF from Kiremidjian and Shah to Seed et al, soil class = 0, damping = 5%.



Reprinted from "Probabilistic Site-Dependent Response Spectra," Kiremidjian, A. S. and Shah, H. C., Journal of the Structural Division, Proceedings of the ASCE, Vol. 106, No. ST1, January 1980, with permission from the American Society of Civil Engineers.

Figure 3-35. Comparison of DAF from present study to DAF from Seed et al, soil class = 1, damping = 5%.

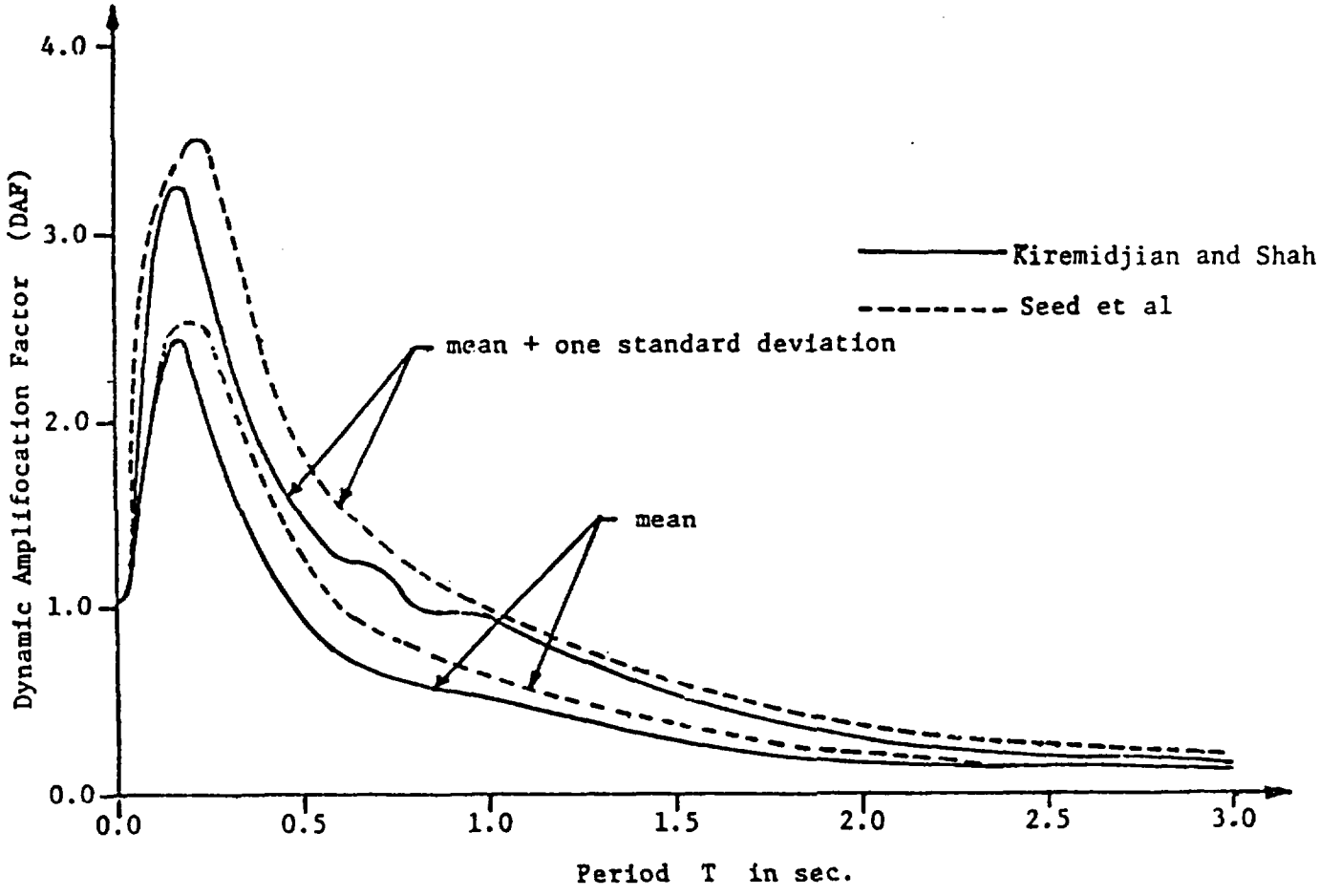


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Figure 3-36. Comparison of DAF from present study to DAF from Seed et al, soil class = 2, damping = 5%.

velocity effects (low attenuation). A representation of this velocity propagation effect is given by the ATC 3-06 Spectra for the appropriate seismic areas of the Eastern United States. This will be shown in paragraph 3-8. All shape factor effects are summarized in figure 3-37. When selecting a design earthquake spectrum, the engineer will consider which of these factors are applicable to the site and how the resulting DAF shape may coincide with the dynamic frequency characteristics of the structure.

g. Formulation of effective response spectra. For the cases where the ATC3-06 method of paragraph 3-8 is to be supplemented or replaced by special site information (para 3-3 to 3-6, and perhaps a site response analysis such

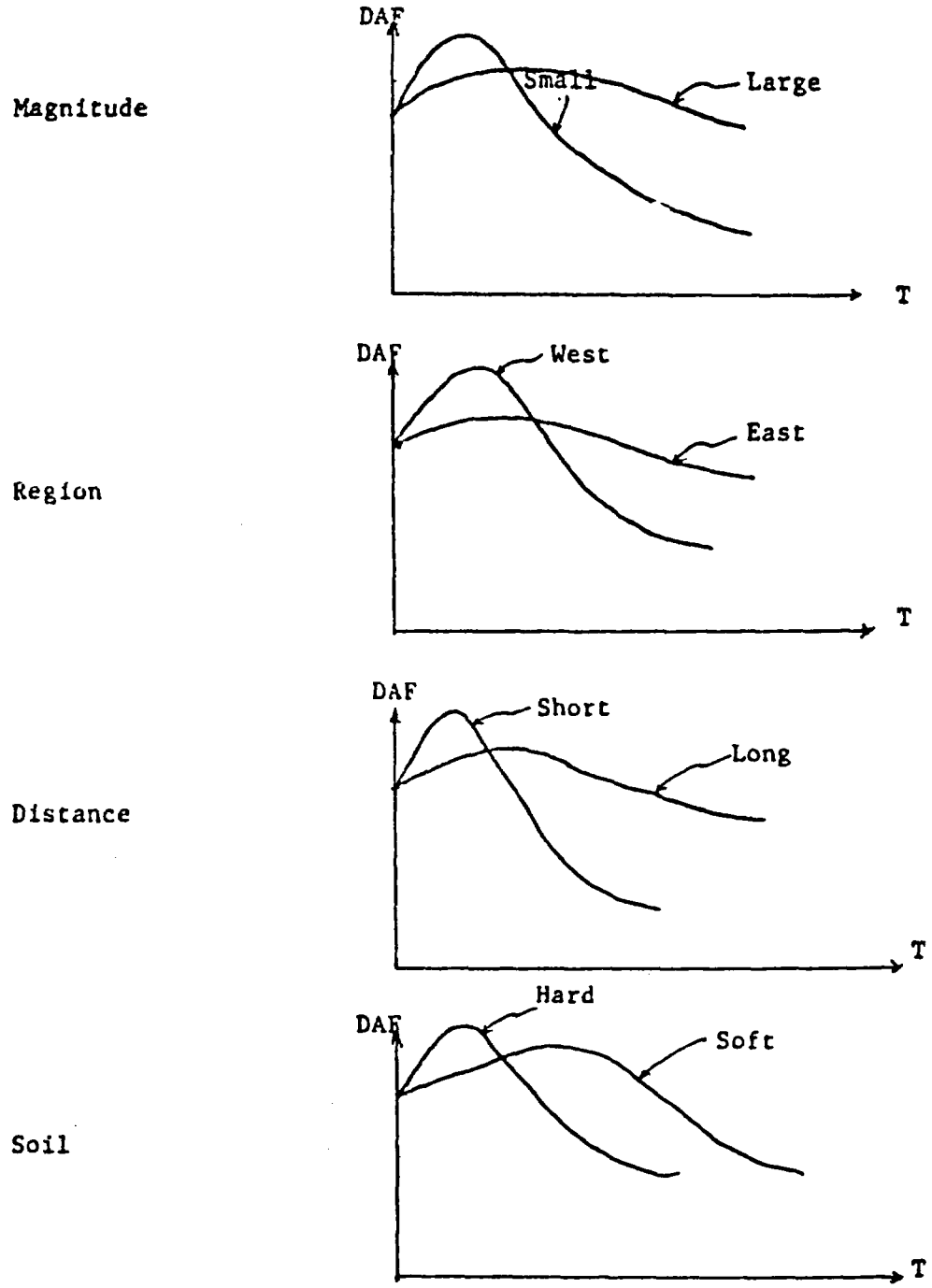

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Figure 3-37. Factors effecting spectral shape.

as SHAKE (Biblio 54), then the mean (or median) spectral shape (DAF) will be used. The specified site spectrum will be mean (or median) PGA times mean (or median) DAF. The mean DAF values for site soil conditions are given by Seed, (Biblio 56) and Kirimedjian and Shah, (Biblio 33) and these may be supplemented by the results from a site response analysis. For example, if one or more spectra are available from a site response analysis, and or if any actual recorded event spectra are judged to be appropriate for the site, then these spectra will be normalized to the mean (or median) PGA value and averaged with the empirical mean PGA X DAF shape. The averaging should be based on a weighted judgement of the relative quality and applicability of the available spectral information, see paragraph C-3*f*, appendix C.

h. Effective response spectra. In paragraph 3-8*e*, there is a discussion of the concept of an effective response spectrum where high frequency (short period) response peaks are reduced to represent the absorption or filtering effect of the actual building size on the short spikes of ground acceleration input. For the case of the mean PGA times mean DAF specified in paragraph 3-6*g*, it is assumed that this mean spectrum is the effective response spectrum; the mean DAF represents both a smoothed or reduced peak shape in the short period range and an average conservative envelope of near and far event ground motion response in the longer period range. It therefore provides for the same effects as discussed in paragraph 3-8*e*.

3-7 Interpretation and summary.

The various concepts and methods of specifying ground motion have been presented. This paragraph provides discussions of the uncertainty in forecasted values; and the relation of selected levels of ground motion to design criteria.

a. Recognition of uncertainty in forecasted values. Each step in the specification of site ground motion involves uncertainties due to empirical relations fitted to limited data; varying assignment of values to general measures of magnitude, intensity, and source-to-site distance; and varying expert opinions. These individual uncertainties have been discussed in the appropriate paragraphs dealing with each parameter necessary for the ground motion forecast. It is intended to assemble these uncertainty measures so as to describe the total reliability of a specified site ground motion.

(1) *Site severity (PGA).* Given the acceptable hazard in terms of the return period (T_R) for the exceedence of structural performance criteria (elastic design level or functional level) the corresponding site severity parameter (PGA) is derived from the following measures of seismicity and attenuation:

—Site to source distance (R), for the one or more sources capable of producing the PGA at the site.

—Magnitude or source-intensity (M or I_o) necessary to produce PGA at the site.

—The appropriate attenuation relation for the geotectonic region and site conditions, and the relation of PGA to site-intensity.

—The probability model and combinatorial procedures required for the evaluation of the PGA corresponding to a given return period T_R.

The resulting forecasted PGA is subject to the uncertainties in the above listed measures. It can be represented as an estimated mean (or median) value of PGA. This forecasted mean PGA is scattered about the true mean PGA (corresponding to a given return period) with an estimated (sampling error) coefficient of variation V_p equal to about 10 to 20 percent.

(2) *The envelope quality of a statistical DAF.* The primary source of spectral shape or DAF information is by the statistical averages of records from common general categories of distance (R), magnitude (M), and soil conditions (S). However, in order to have a sufficient sample size, there is rather wide variation in the individual record conditions (R, M, S) within any general category. This individuality causes a large contribution to the coefficient of variation V_{DAF} of the DAF; but in terms of forecasting future ground motion, it has the following useful interpretation. Referring to figure 3-38, the possible single events at a site can have (for example) either condition "A" or "B"; corresponding to large magnitude and near source ("A") or moderate magnitude and far source ("B"). The average envelope curve would therefore be exceeded only in the case where the actual event conditions are not enveloped by this upper curve. Thus, in this example, for periods less than T_1, the envelope is much more conservative if conditions "B" were to occur. The chance that this curve will be exceeded by the actual future event DAF is the chance of having both conditions "A" and structural period less than T_1. This would be the product of the two probabilities of (condition "A") and of (period $T \leq T_1$), and would be small. As a rough, but

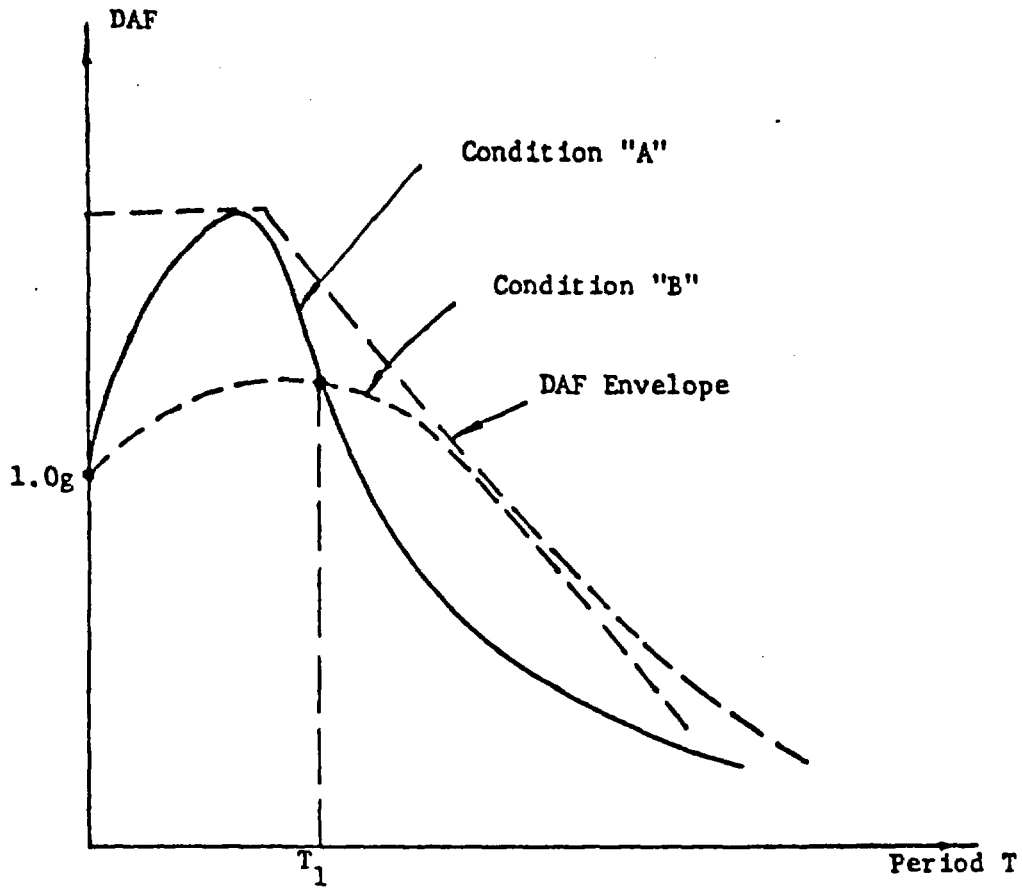

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Figure 3-38. Envelope quality of the DAF shape.

reasonable value, it is assumed that the combined envelope shape would be approximately equivalent to a 90 percent confidence interval for a single event spectrum. Therefore, in order to best represent the fact that the envelope curve and its simplified design DAF version is an envelope of many possible future event conditions, the design DAF in the next paragraph (3) is assumed to be equivalent to 90 percent confidence limit.

(3) *Smoothed or simplified design DAF.* The mean or median value DAF results directly from the statistical average of the normalized (DAF) values from the site-representative earthquake records. The common range of coefficients of variation is 0.3 to 0.5. However, when the mean or median DAF values are smoothed and simplified to provide a design DAF (see ATC 3-06), the final shape represents an envelope for any of the possible spectral shapes that could occur at the site. Because of this necessity for the simplified envelope in order to provide a practical input (without steep peaks and valleys) for dynamic analyses, it is not possible to describe the design DAF in terms of a central value and coefficient of variation. It is estimated that the design DAF represents at least a 90 percent upper confidence limit on the true DAF that could occur at the site; or in terms of probability, the probability that a future event DAF would exceed the design DAF is about 10 percent.

b. Reliability of specified ground motion. The classical hazard analysis (STASHA) provides a central PGA value for a given return period or risk of exceedance. Due to prediction error, the true PGA for the given return period has a 50 percent chance of exceeding this central PGA.

(1) Then, with the recognition that the DAF shape is a conservative envelope of DAF's from near and far events, and assigning a very rough judgemental probability of 10 percent that the DAF of any single event would exceed the envelope shape, the reliability of the effective design spectrum (PGA)(Design DAF) is given by,

$$1 - (0.5)(0.10) = 0.95 \text{ or } 95\%$$

While this reliability measure is based on very subjective measures of uncertainty, it provides a reasonable description of the actual conditions. In summary, given an accepted return period for the forecasted ground motion, there is only a 5 percent chance that the design spectrum would be exceeded. The STASHA program offers a more rigorous and complete method of establishing the reliability by means of its Bayesian Hazard Analysis option, see appendix D.

(2) The effects or consequences of uncertainties and variabilities in specified ground motion values for a site are best evaluated after the consideration of the total structural design process in chapter 4. When the forces and deformations in the structural model have been evaluated for the specified ground motion, then judgements can be made concerning the effect of seismic input variations on the performance of the final design. For example, if critical members have high levels of inelastic demand, and if reasonable variations in input can increase this demand beyond the failure threshold, then the designer should strengthen or modify this part of the structure.

c. Site specific hazard curves. Hazard is defined as the probability of exceeding a given level of site PGA during a given exposure time t, and where PGA is the forecasted mean or median value from the hazard analysis. This central forecasted PGA value is the measure of ground motion severity and is used (in step V) as the spectral scaling factor for the site response spectrum

$$S_a = PGA \times DAF,$$

where the DAF is a reliable envelope shape for all of the spectral shapes that could be produced by the events capable of generating the PGA at the site. Because there may be more than one source and (or) more than one possible earthquake event at different locations on a source, it is not possible to calculate directly the value of a PGA having a specified hazard or probability of exceedence. Several values of hazard $P[PGA > PGA_j]$ must be evaluated for given incremented values of PGA_j, and then a hazard curve is constructed through the plot of the hazard versus PGA_j points; a hazard curve is shown in figure 3-39. With this curve it is possible to determine the site PGA value corresponding to a specified hazard value for a given exposure time: for example, the PGA_I for EQ-I having a 50 percent chance of exceedence in 50 years.

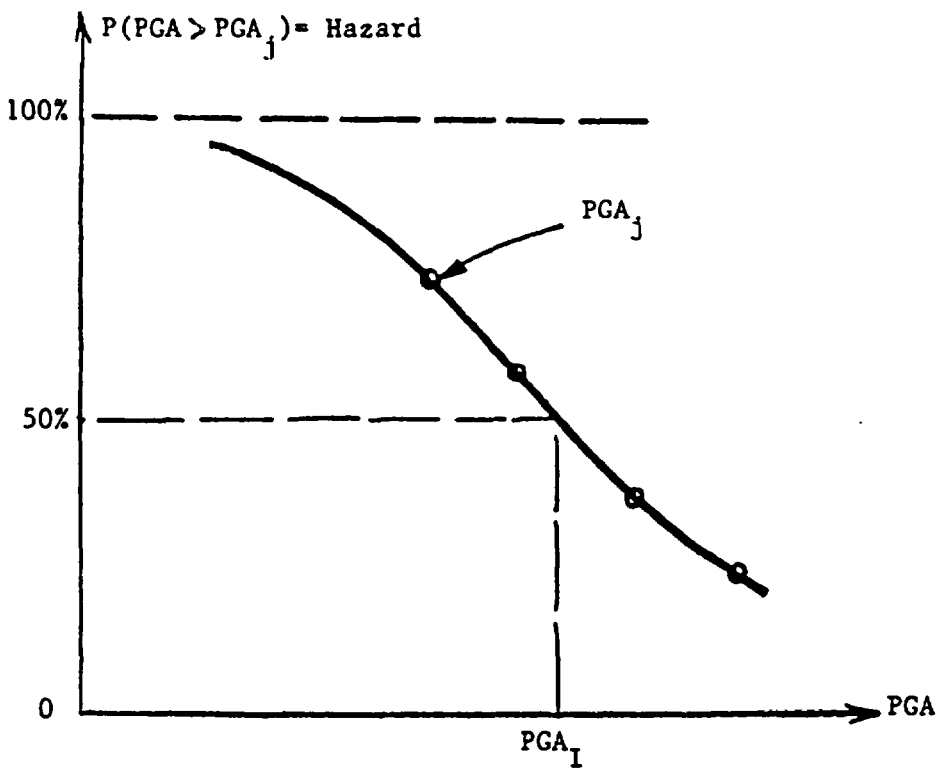

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Figure 3-39. Hazard curve for site PGA with exposure time of 50 years.

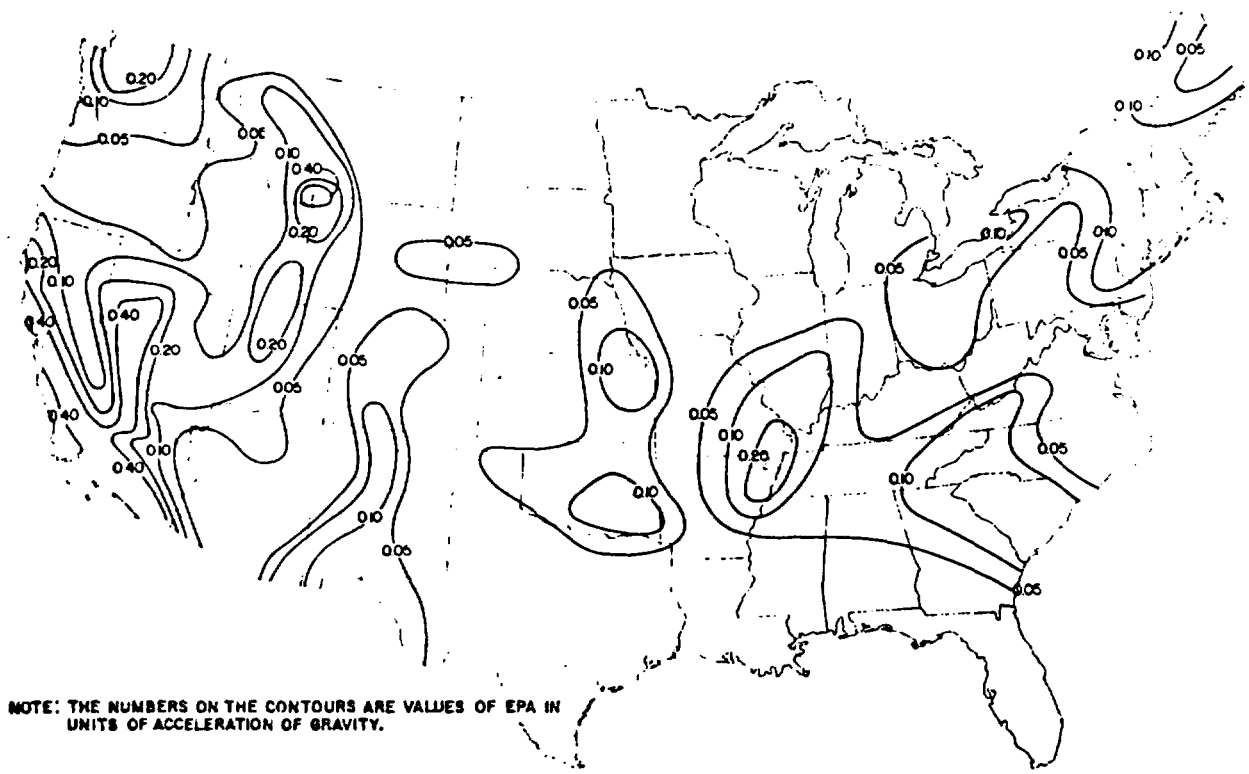
In appendix D, a simplified example 1 shows the individual steps necessary to calculate one of the incremental hazard curve points $PGA_j = 0.20g$ for a 50 year exposure time. The other examples, 2 and 3, of appendix D show the more detailed procedures using the STASHA computer program as required for the practical evaluation of the hazard at a given site.

Section III. THE ATC3-06 METHOD

3-8 The ATC3-06 method.

This method as documented in ATC3-06 (National Bureau of Standards, Special Publication 510) and as prescribed in this paragraph will be used according to the guidelines in figure 3-1. The resulting design spectra are to be considered as the minimum seismic loading criteria. Where there are exceptional site conditions such as close source proximity, or highly responsive soil columns, or if the configuration or use of the structure is very different or special, then the hazard analysis methods in paragraphs 3.1 to 3.7 are to be used to supplement these minimum criteria. Any changes from these criteria are subject to approval by the reviewing agency.

a. Determination of site severity. For a given site location the contour maps, figures 3-40 to 3-43 provide the basis for evaluating the site severity or scaling factors for EQ-1 and EQ-22. These figures provide contour values A_a and A_v having a 10 percent probability of exceedence in 50 years. Definitions of A_a and A_v are given in figure 3-44. Figure 3-45 gives curves that convert the contour values to the A_a or A_v values corresponding to the probabilities of exceedence for EQ-I (50% in 50 years) and EQ-II (10% in 100 years). The value for EQ-I is found where the contour level curve intersects the 50% probability line for 50 years. The value for EQ-II is found where the contour level curve intersects the 10% probability line for 100 years.



NOTE: THE NUMBERS ON THE CONTOURS ARE VALUES OF EPA IN UNITS OF ACCELERATION OF GRAVITY.

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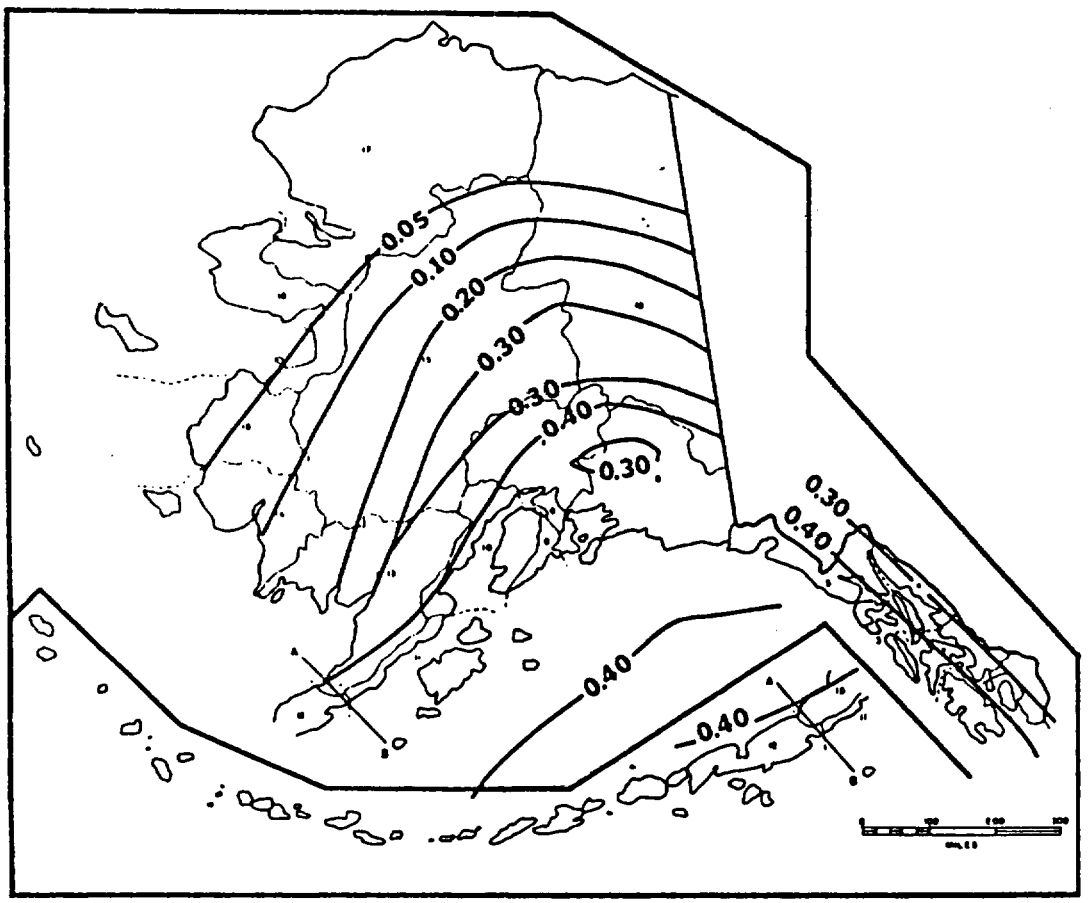
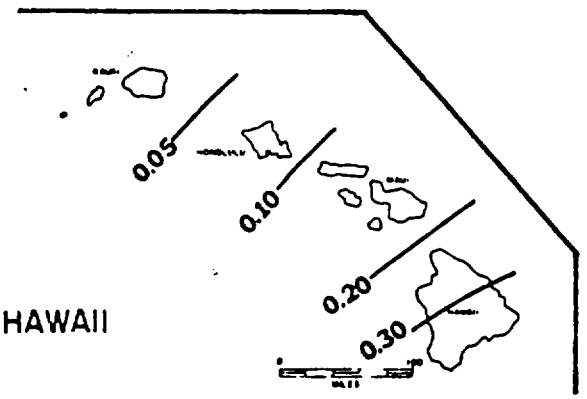
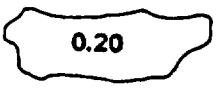
Figure 3-40. Contour map for effective peak acceleration.



ALASKA



HAWAII



PUERTO RICO

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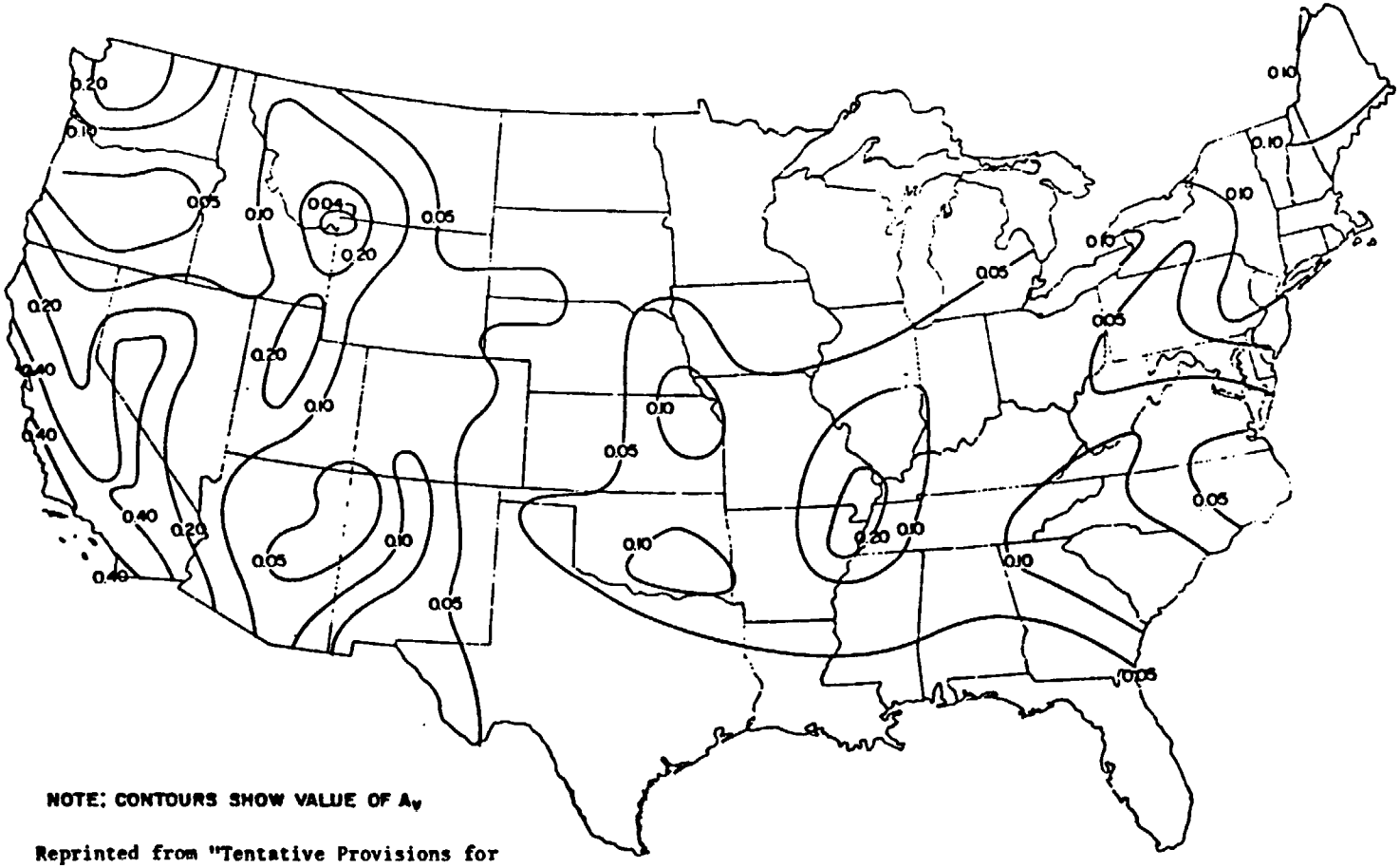
Figure 3-41. Contour map for effective peak acceleration.



NOTE: CONTOURS SHOW VALUE OF A_v

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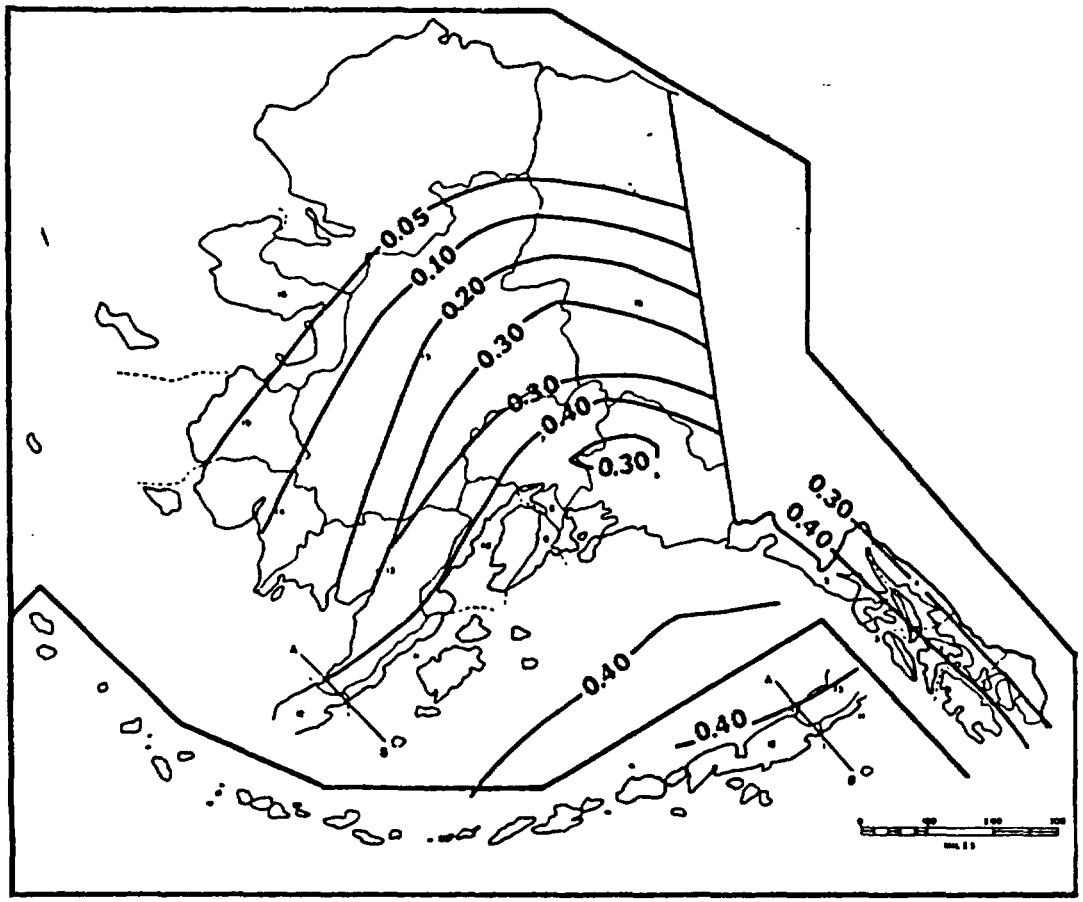
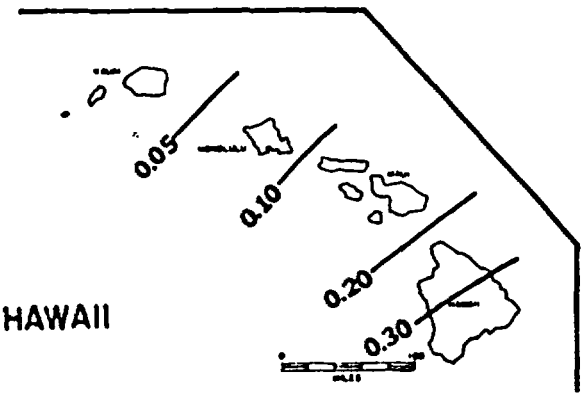
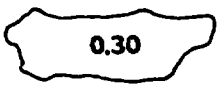
Figure 3-42. Contour map for effective peak velocity-related acceleration coefficient.



ALASKA



HAWAII



PUERTO RICO

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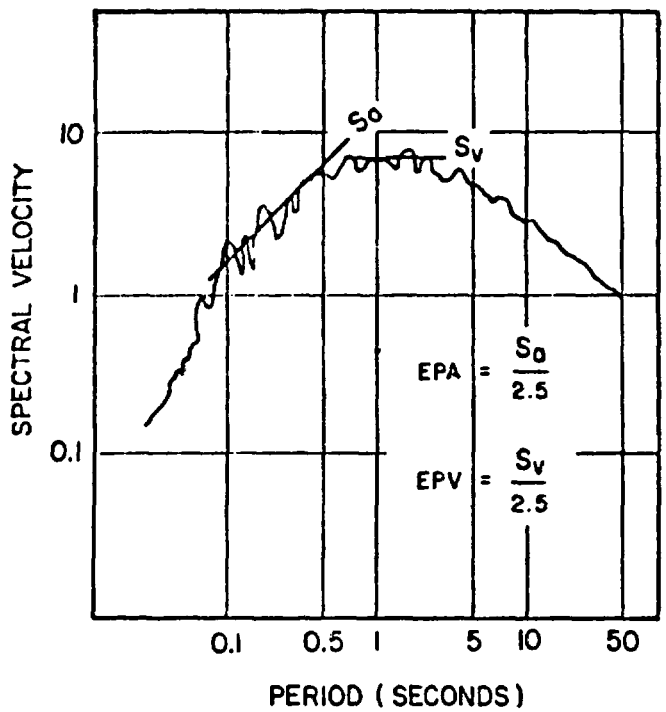
Figure 3-43. Contour map for effective peak velocity-related acceleration coefficient.



Map values of A_a = EPA in g's

Map values of A_v = EPV/30 in g's, and EPV is in inches/sec.

A_v is the velocity related acceleration value.

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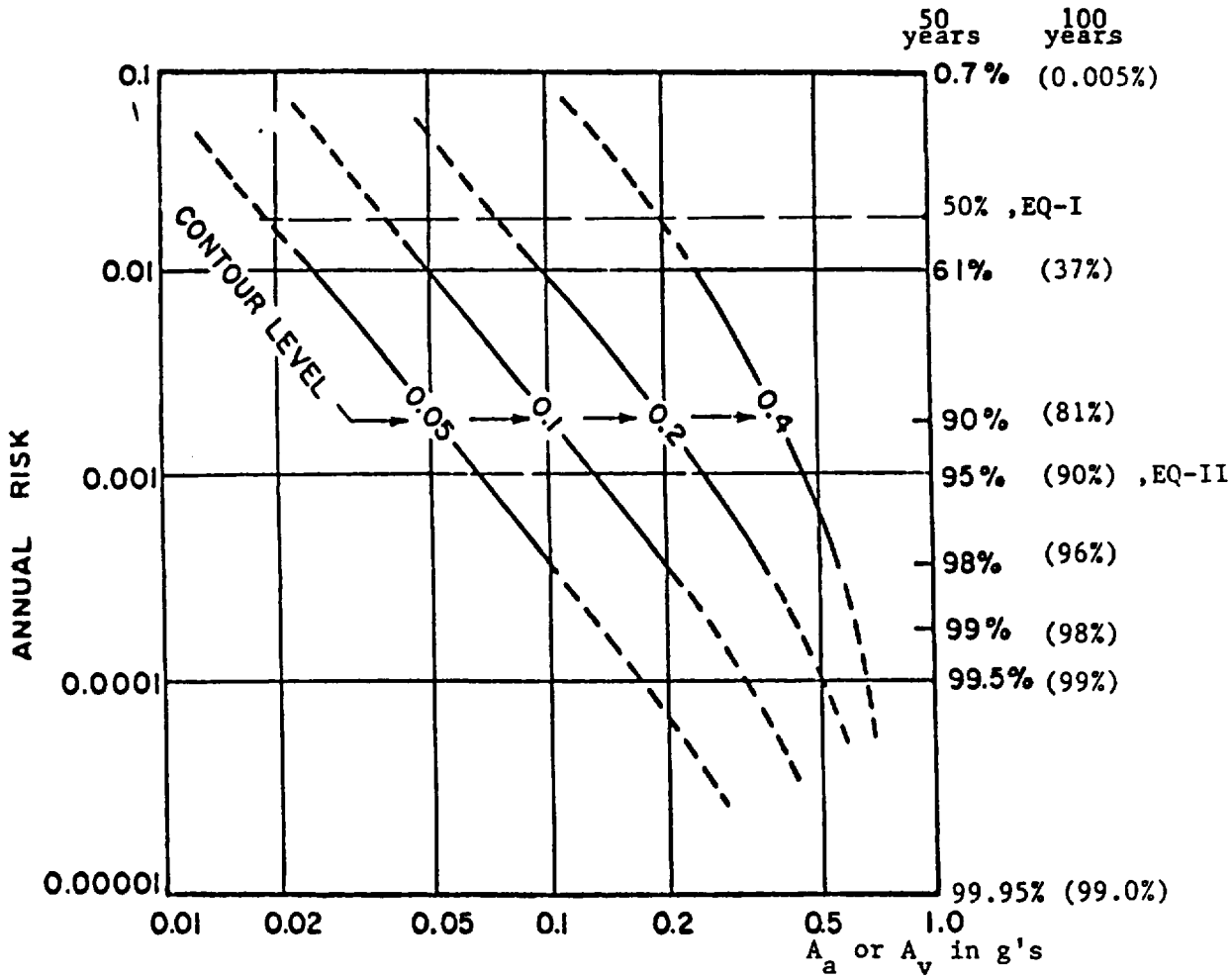
Figure 3-44. Schematic representation showing how effective peak acceleration and effective peak velocity are obtained from a response spectrum.



Note: axis on right provides probabilities of non-exceedence in exposure times of 50 years and 100 years.

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Figure 3-45. Annual risk of exceeding various effective peak accelerations for locations on the indicated contours of A_a and A_v in figures 3-40 to 3-43.

(1) Table 3-4 gives a summary of the resulting A_a and A_v values for each corresponding map contour values. These A_a and A_v values are to be used to scale the response spectrum shape DAF as per equations 3-27 to 3-30 in paragraph 3-8c.

(2) Note in figure 3-45, that any 100 year probability of non-exceedence can be obtained by the square of the corresponding 50 year probability; the occurrence of two successive 50 year periods of non-exceedence.

(3) Also, even though figure 3-45 was originally meant to be used for EPA = A_a values in ATC3-06, the stated probability values are assumed here to be also applicable to the A_v values, such that both A_a and A_v can be converted to the EQ-I and EQ-II probability values, at all locations in the United States. This assumption is considered valid because any A_v value is derived from the A_a value at a given map location and therefore has the same probability value as the A_a.

b. Determination of site soil type. The site soil profile type will be determined and identified as S_1, S_2, or S_3 according to the definitions given in table 3-5.

Table 3-4. Map contour and ground motion levels.

ATC 3-06 Map Contour Level A_a or A_v in units of g (figs 3-40 to 3-43)	Design Ground Motion Level A_a or A_v and Probability of Exceedance*	
	EQ-I (50% in 50 years)	EQ-II (10% in 100 years)
0.05	0.02	0.06
0.10	0.04	0.12
0.20	0.08	0.25
0.40	0.20	0.45

* For use in equations 3-27 to 3-30

Table 3-5. Site soil profile types.

SOIL PROFILE TYPE S_1 is a profile with:

1. Rock of any characteristic, either shale-like or crystalline in nature. Such material may be characterized by a shear wave velocity greater than 2,500 feet per second, or
2. Stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE S_2 is a profile with deep cohesionless or stiff clay conditions, including sites where the soil depth exceed 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE S_3 is a profile with soft-to-medium-stiff clays and sands, characterized by 30 feet or more of soft-to-medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, Soil Profile S_2 shall be used.

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c. Determination of the design response spectra. With the known values of A_a and A_v for EQ-II and the site soil type (S_1, S_2, or S_3), the 5 percent damped, EQ-II acceleration response spectrum is given by the following equations; note that these equations specify constant levels of spectral acceleration S_a, spectral velocity S_v, and spectral displacement S_d, within the prescribed ranges of structural period T (refer to the spectrum relations given in fig 3-44 and in para C-2*b* of appendix C).

For $T \leq 4$ seconds:

$$S_a = \frac{1.22}{T} A_v S_i \text{ g's,} \quad \text{(constant } S_v = 75 A_v S_i \text{ in/sec)} \qquad \text{(eq 3-27)}$$

but always less or equal to

$$S_a = 2.5 A_a \text{ g's} \quad \text{(constant } S_a \text{ in g's.)} \qquad \text{(eq 3-28)}$$

and

$$S_a = 2.0 A_a \text{ g's when } S_i = 1.5 \text{ and } A_a \geq 0.30 \qquad \text{(eq 3-29)}$$

For $T > 4$ seconds:

$$S_a = \frac{4.88}{T^2} A_v S_i \text{ g's} \quad \text{(constant } S_d \ \frac{150}{\pi} A_v S_i \text{ inches)} \qquad \text{(eq 3-30)}$$

Values for S_i are given in table 3-6. These equations for S_a are equivalent to the constant acceleration, velocity, and displacement levels shown on the general tripartite, logarithm scale graph in figure 3-46. A specific example is shown for $A_a = A_v = 0.40$ in figure 3-47. Note that equation 3-27 differs in form from that of the base shear equation

$$V = \frac{1.2 A_v S_i}{R T^{2/3}}$$

given in the ATC 3-06 document. The 1.2 value is a round-off of the 1.22 value in equation 3-27, the $T^{2/3}$ exponent value allows the base shear equation to represent multi-mode response effects. The base shear equation is for the equivalent static force method at a single period value and needs this empirical method of allowing for the combination of response from all modes.

(1) The 5 percent damped EQ-I Spectrum is equal to the EQ-II spectrum multiplied by the ratio of the EQ-I to the EQ-II values given in table 3-4. Linear interpolation may be used for values between those given in this table.

(2) The flat plateau for S_a as given by equation 3-28 or equation 3-29 provides a conservative (high) representation of response for the higher (higher than first mode) modes of structural response where the modal periods are less than 0.2 or 0.3 seconds. However, this conservative response measure may be excessive for a Soil Profile Type S_3. Referring back to figure 3-30 of paragraph 3-6, the corresponding soft to medium clay and sand site condition has a mean spectral shape that rises from the zero period value to the plateau at about 0.3 seconds. Higher modes can have periods below this value and therefore would have S_a values lower than the flat plateau. Following the recommended relation given in the commentary of chapter 5 in the ATC3-06 (National Bureau of Standards, Special Publication 510); for the case of Soil

Table 3-6. Soil profile coefficient.

Soil Profile Type

	S_1	S_2	S_3
S_i =	1.0	1.2	1.5

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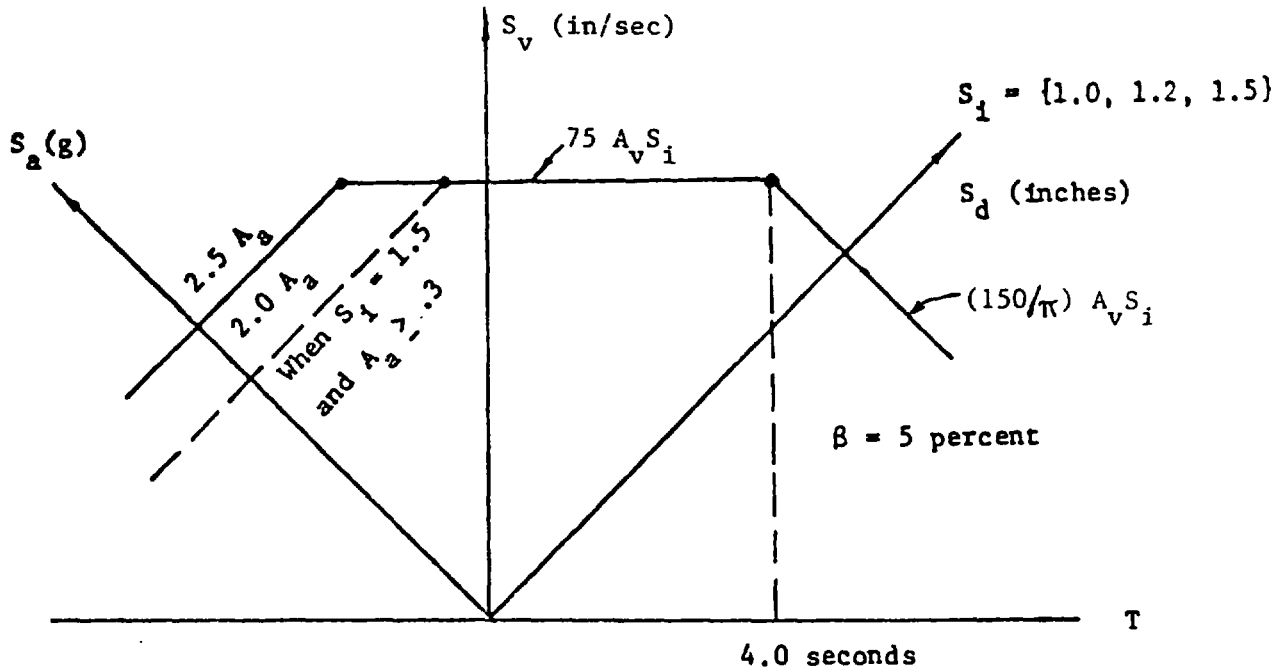

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Figure 3-46. Tripartite representation of EQ-II.

Profile Type S_3, and for modes higher than the first mode, the S_a values may be determined from a straight line extending from the A_a value at zero period to the plateau at period equal to 0.3 seconds.

d. Consideration of structural damping ratio. All of the design spectra given in paragraph 3-4c are for structural damping equal to 5 percent of critical damping. These spectra may be converted to other damping ratios by use of the factors given in table 3-7. Linear interpolation may be used to provide factors for intermediate damping values. The factors in this table are based upon empirical relations given by Newmark and Hall, (Biblio 44). The median spectral shape given in this Biblio (44) is sufficiently close to the shape in this paragraph, so that the damping relations are applicable. The table 3-7 factors represent rounded-off average of the Newmark values for the constant acceleration plateau and the constant velocity (1/T) range of the spectral shape. Since the specified spectra in this paragraph are formed by various simplified factors such as the ($2.5A_a$) effective plateau, and the soil type coefficients (S_1, S_2, S_3), the rounded-off average damping factors in table 3-7 are judged to be consistent with these other factors. If more accurate values are desired, then the Newmark and Hall relations may be used for the median spectral shape, Biblio 44).

e. Representation of the effective response spectrum. In regions of strong seismicity, and for site locations near to sources, the response spectra from the single possible events (producing the same site PGA) can either have a high frequency peak shape for near events, or have a more constant shape at lower frequencies for a distant large event, see figure 3-48. The ATC3-06, spectral shape provides a reliable envelope of the spectra from both near and far events. Further, the horizontal plateau of ($2.5A_v \leq 2.5A_a$) provides the effective structural response spectrum: the high frequency peak response values, usually present in near-source records and spectra would be filtered out by the structure size, mass, and foundation configuration, and actual structure response is represented by the plateau level in this high frequency range. Note that the PGA at the site is the same for each (near and distant) event. For example, a PGA = 0.60g may correspond to the ATC3-06 map contour value of A_a = 0.40g. It is important to recognize that the EPA = A_a = 0.40g = 2/3 (PGA = 0.60g) applies to the effective spectrum plateau in the high frequency range; the remainder of the spectral envelope corresponds to the site severity as represented by the forecasted central PGA value.

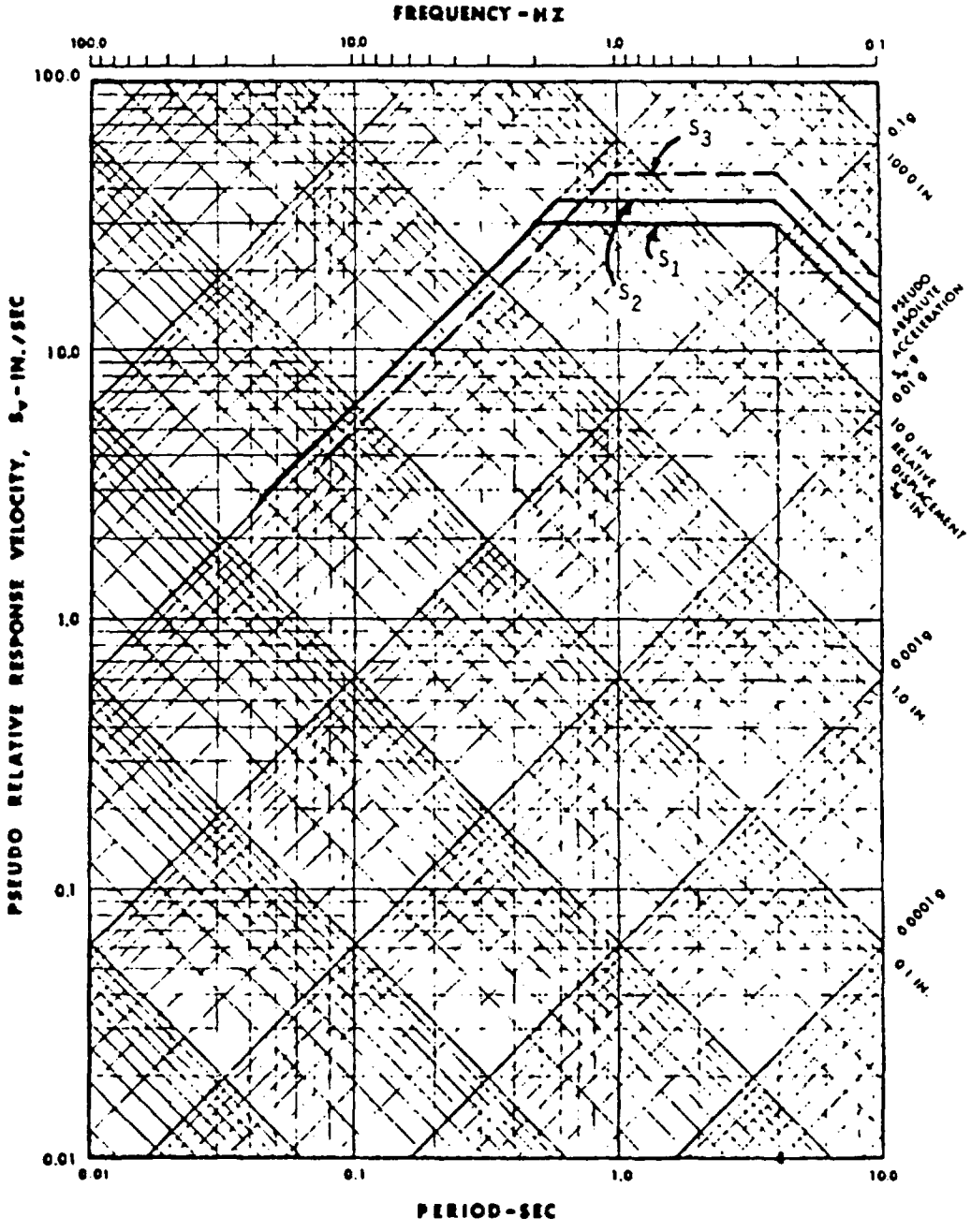

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Figure 3-47. EQ-II spectra for $A_a = A_v = 0.40$, and $\beta = 5\%$.

Table 3-7. Damping adjustment factors.

β Percent	Multiplying Factor for the 5 Percent Spectrum
2	1.25
5	1.00
7	0.90
10	0.80
15	0.70
20	0.60

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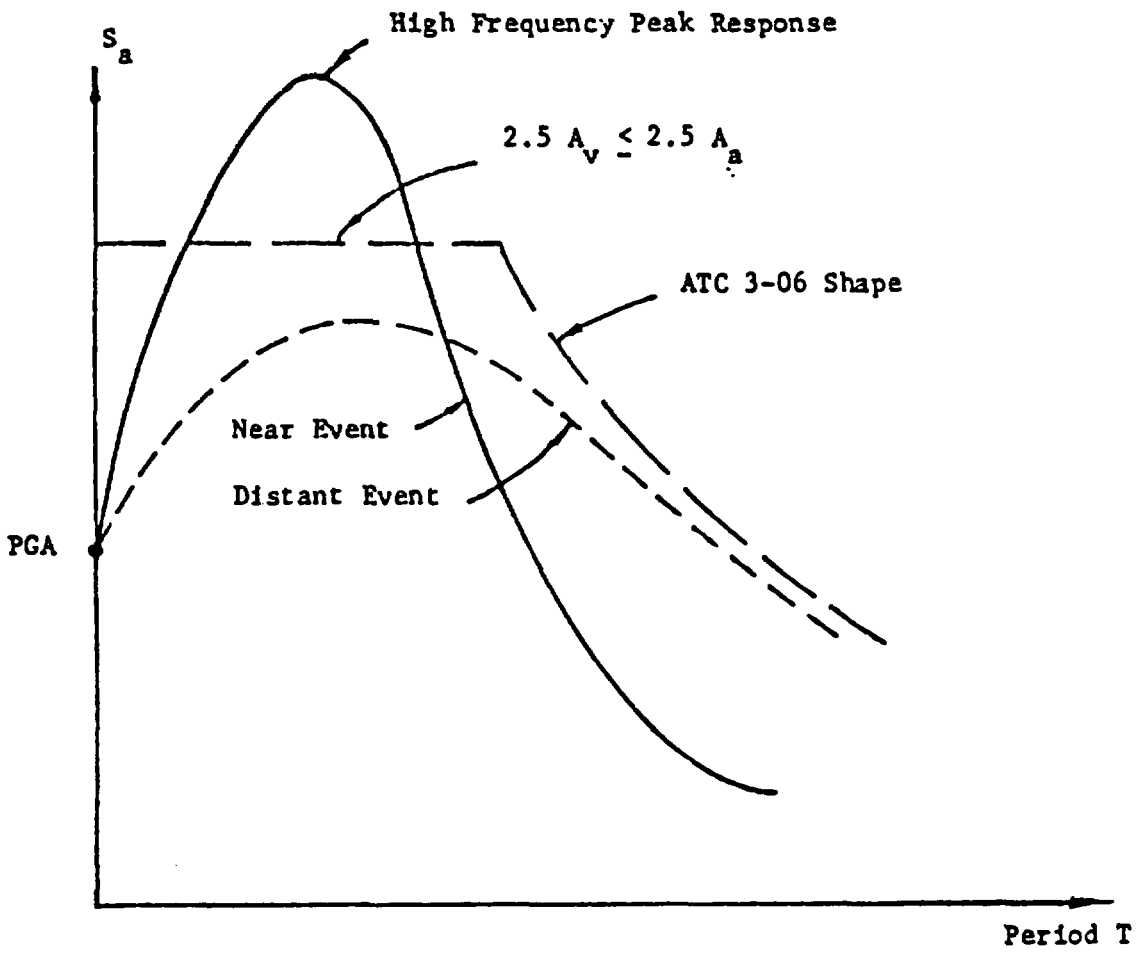

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Figure 3-48. Effective spectral envelope.

f. Representation of regional attenuation differences. The ATC3-06 contour maps provide A_a and A_v values, and the spectral plateau rule requiring $2.5\ A_v \leq 2.5\ A_a$ is a simple yet effective method of representing the low attenuation rate of ground motion in some areas of the EUS and WUS, see figure 3-49. When the map gives $A_v \geq A_a$, then the plateau value of $2.5A_a$ extends further on the period scale and gives a spectral shape having larger values in the moderate frequency range. This represents a preservation of the amplitude of moderate frequency ground motion components, or a low attenuation of these components which is characteristic of the wave propagation in the EUS and in some regions of the WUS outside of California.

g. Examples using the ATC3-06 method.

(1) *Site location.* Las Vegas, Nevada. Soil type S_2; S = 1.2 from table 3-6.

(a) Find Map Contour Values:

figure 3-40, A_a = 0.10

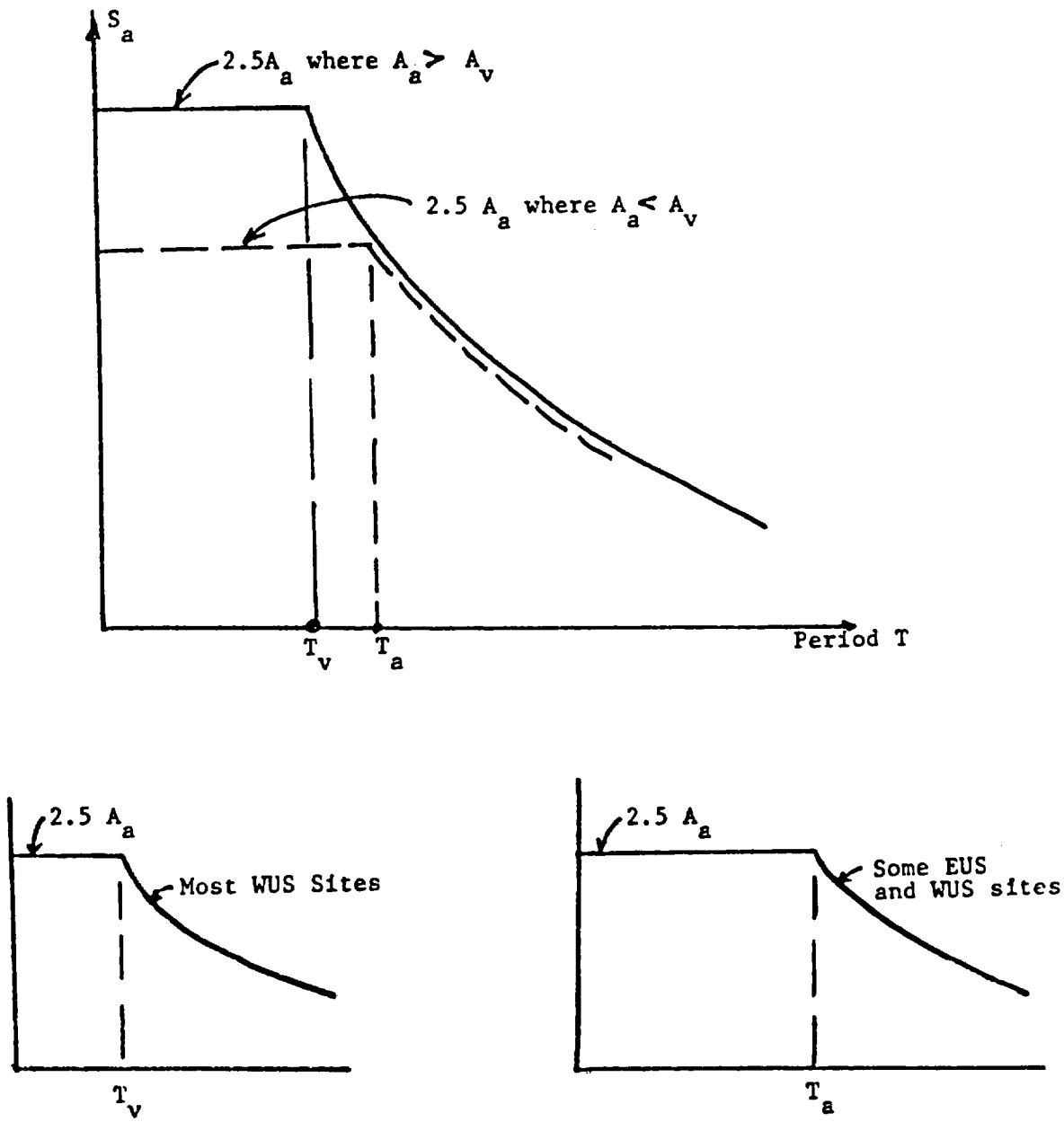

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Figure 3-49. Regional shape difference.

figure 3-42, $A_v = 0.15$

(b) Obtain Special Scaling Factors from Table 3-4:

Using Interpolation,

EQ-I, $A_a = 0.04$, $A_v = 0.06$
EQ-II, $A_a = 0.12$, $A_v = 0.18$

(c) The specified Structural System Damping Values are given as 5 percent for EQ-I and 10 percent for EQ-II. Table 3-7 provides Damping Adjustment Factors of 1.00 for $\beta = 5$ percent and 0.80 for $\beta = 10$ percent. Using these damping factors, the Acceleration Response Spectra are given by equations (3-27) and (3-28) for $T \leq 4$ seconds.

EQ-I

$S_a = (1.22/T)A_vS_i$ X Damping Adjustment Factor,
$= (1.22)(0.06)(1.2)(1.00)/T$
$= (0.0878/T)g$,

but always less or equal to

$S_a = 2.5A_a$ X Damping Adjustment Factor
$= 2.5(0.04)(1.00)$
$= 0.10g$

EQ-II

$S_a = (1.22)(0.18)(1.2)(0.80)/T$
$= (0.211/T)g$

but always less or equal to

$S_a = 2.5(0.12)(0.80)$
$= 0.24g$

These EQ-I and EQ-II Spectra are shown in figure 3-50.

(2) *Site location.* Emeryville, California. Soil Type S_3; $S_i = 1.5$ from table 3-6.

(a) Find map contour values:

figure 3-46, $A_a = 0.40$
figure 3-48, $A_v = 0.40$

(b) Obtain Special Scaling Factors from table 3-4.

EQ-I, $A_a = 0.20$, $A_v = 0.20$
EQ-II, $A_a = 0.45$, $A_v = 0.45$

(c) The specified structural system damping values are given as 5 percent for EQ-I and 7 percent for EQ-II. Table 3-7 provides Factors of 1.00 for $\beta = 5$ percent and 0.90 for $\beta = 7$ percent. Using these factors with equations (3-27) and (3-29) for $T \leq 4$ seconds.

EQ-I

$S_a = (1.22/T)A_vS_i$ X Damping Adjustment Factor
$= (1.22)(0.20)(1.5)(1.00)/T$
$= (0.366/T)g$,

but always less or equal to

$S_a = 2.0A_a$ X Damping Adjustment Factor
$= 2.0(0.20)(1.00)$
$= 0.40g$

EQ-II

$S_a = (1.22)(.45)(1.5)(0.90)/T$
$= (0.741/T)g$

but always less or equal to,

$S_a = (2.0)(0.45)(.90)$
$= 0.81g$

(3) These EQ-I and EQ-II Spectra are shown in figure 3-51. Note that the higher mode transition spectrum shape is shown for each spectrum in the zero to 0.3 second period range.

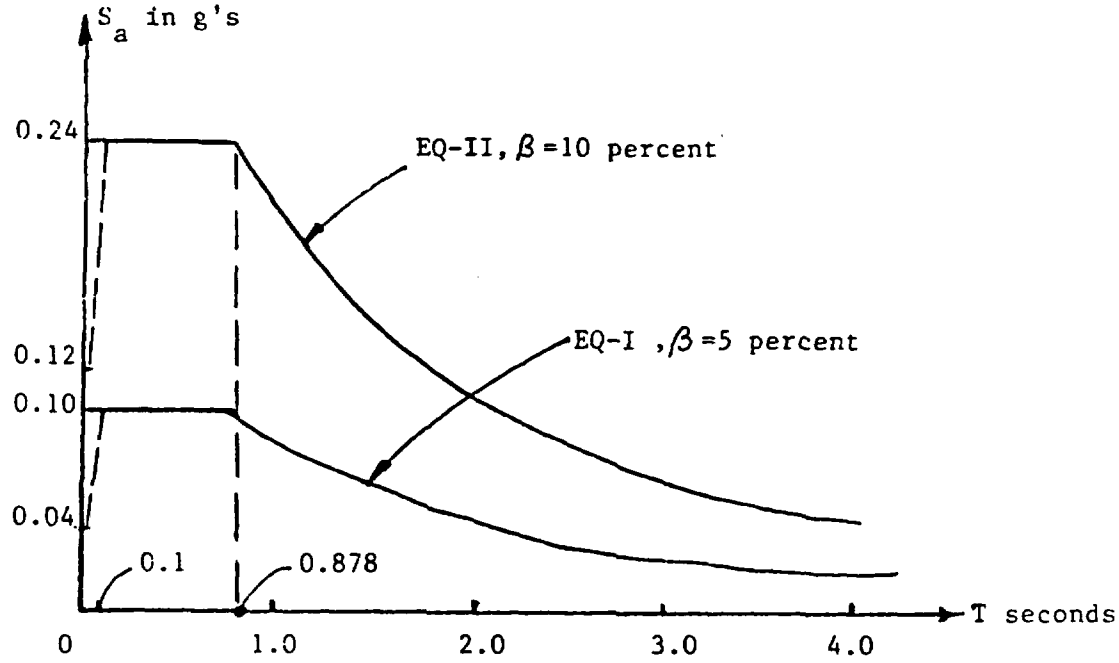

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Figure 3-50. Las Vegas, Nevada site spectra for soil type S_2.

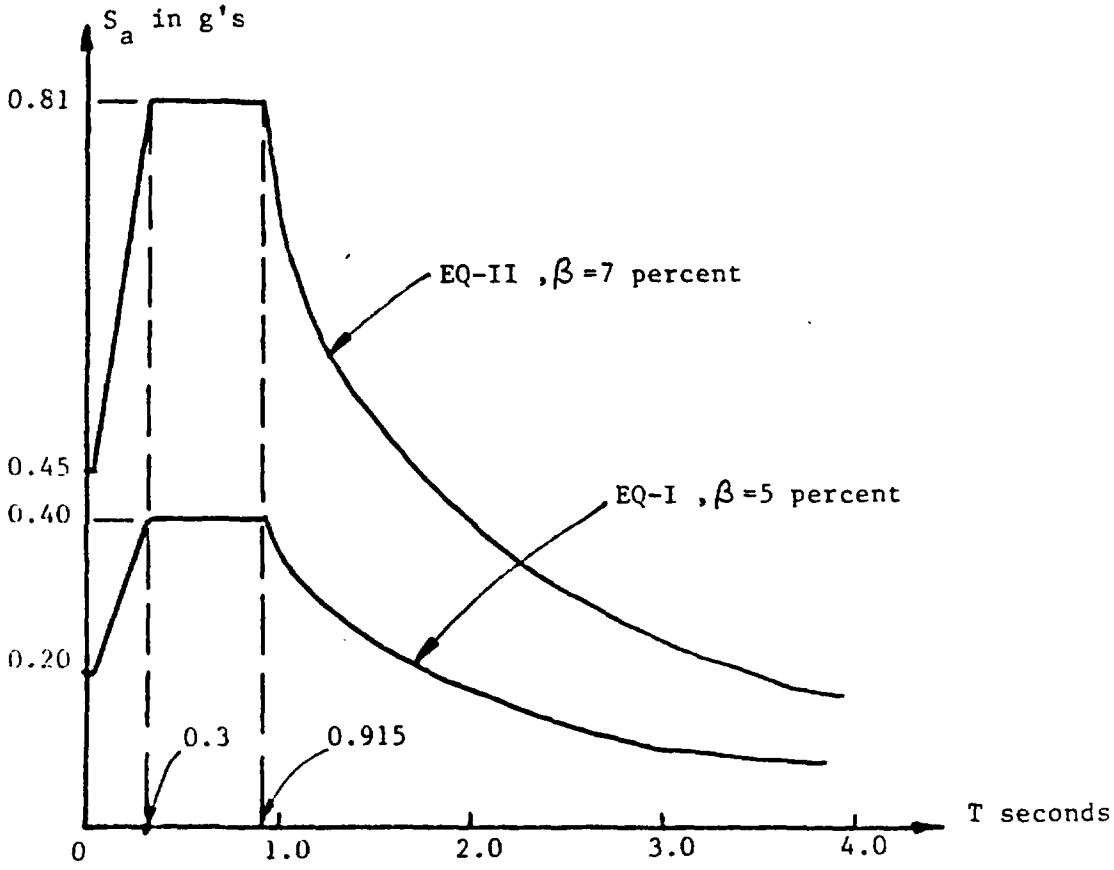

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Figure 3-51. Emeryville, California site spectra for soil type S_3.

CHAPTER 4
CRITERIA FOR STRUCTURAL ANALYSIS

4-1. Introduction.

This chapter prescribes the dynamic analysis criteria for the development of a seismic-resistant structural concept, the determination of the seismic forces to be applied to the structure, and the design and analysis of structural members and connections. The criteria and design standards for the dynamic analysis approach herein, for the seismic design of buildings, will be used only when directed or approved in lieu of the lateral static forces procedure of the Basic Design Manual. The procedures to determine effective response spectra for selected risk levels and site conditions are developed in chapter 3 (e.g., fig 3-3). This chapter provides the structural performance requirements for the selected risk levels in accordance with paragraph 3-3*b*.

a. Essential facilities. Criteria set forth in this chapter have been developed primarily for the design of essential facilities, as classified in paragraph 1-1*d*, that are assigned an I-factor equal to 1.5 in the Basic Design Manual.

b. High-risk structures. Criteria set forth in this chapter may be applicable to the design of high-risk buildings, as classified in paragraph 1-1*d*, that are assigned an I-factor equal to 1.25 in the Basic Design Manual.

c. All others. Applicable portions of criteria set forth in this chapter may be used as a means for considering the dynamic characteristics of irregular structures or framing systems to comply with the Basic Design Manual, paragraph 3-3(E)3, and as a means for establishing the lateral design forces and distributions by dynamic analyses to comply with the Basic Design Manual, paragraph 3-3(I).

4-2. General requirements.

a. General. Design and construction will conform to the provisions of the Basic Design Manual if not superseded by or in conflict with the requirements of this manual.

(1) The structural system or type of construction will admit to a rational analysis in accordance with established principles of mechanics and dynamics. A continuous load path, or paths, with adequate strength and stiffness, will be provided to transfer all forces from the point of application to the final point of resistance. The foundation will be designed to accommodate the forces developed or the movements imparted to the building by the design ground motions. In the determination of the foundation design criteria, special recognition will be given to the dynamic nature of the forces, the expected ground motions, and the design basis for strength and ductility of the structure.

(2) Structures will be designed for dead, live, snow, and wind and/or seismic forces as given in the applicable agency manuals and in this manual. Every building or structure and every portion thereof will be designed and constructed to resist the stresses and distortions produced by the dynamic seismic analysis procedure in combination with dead and live loads as specified in this chapter. Where prescribed wind loads govern the design of some or all structural elements, the design analysis will be prepared for both the wind and seismic criteria and the structural elements will be sized for the most severe loading condition.

(3) Stresses and deformations will be calculated as the effect of the dynamic analysis being applied horizontally and coming from any horizontal direction. The effects of vertical accelerations will also be considered in the design of horizontal cantilever and horizontal prestressed components.

(4) Materials and details will conform to the seismic provisions, applicable guide specifications, and criteria herein, including the seismic reinforcing details in the Basic Design Manual. The provisions of this chapter apply to the structure as a unit and also to all parts thereof, including the structural frame or walls, floor and roof systems, anchorages and supports for architectural elements and mechanical and electrical equipment, and other elements.

b. Definitions. Definitions listed in the Basic Design Manual, paragraph 3-3(B), will apply to this manual. Additional definitions are listed in the glossary.

c. Symbols and notations. Symbols and notations listed in the Basic Design Manual, paragraph 3-3(C), will apply to this manual. Additional symbols and notations are listed in appendix A, Symbols and Notations.

d. Dynamic analysis procedure for buildings.

(1) *Essential buildings.* Essential buildings will be designed to resist two levels of earthquake motion. The first level of motion is designated EQ-I and the second and larger amplitude of motion is designated EQ-II. The lateral-force-resisting structural systems of these facilities will be designed to resist EQ-I by elas-

tic behavior as prescribed in paragraph 4-3. The facilities will be evaluated for their ability to resist EQ-II by post-elastic behavior with ductility limitations as prescribed in paragraph 4-4. Guidelines for these dynamic analysis procedures are described in chapter 5.

(2) *High-risk buildings.* Subject to the direction of the approval authority, high-risk buildings will be designed by either of the two following procedures:

(a) Two-level approach. Using two levels of ground motion in accordance with the procedure described in paragraph (1) above, except that the forces resulting from the EQ-I spectral response may be reduced by 15 percent (i.e., use 85 percent of EQ-I responses), unless otherwise directed. The lateral-force-resisting structural systems of these facilities will be designed to resist the modified EQ-I as prescribed in paragraph 4-3.

(b) Single-level design. Using the procedures described in paragraphs (3)(a) or (3)(b) below using an importance factor (I) equal to 1.25.

(3) *All other buildings.* Buildings that are not classified as essential or high-risk facilities will be designed in accordance with one of the following three procedures:

(a) Basic Design Manual criteria with modified seismic force distribution. Determine the distribution of seismic forces in accordance with the modal analysis procedure of paragraph 4-3 with an appropriate response spectrum for EQ-I. Normalize the resulting forces such that the net total seismic shear at the base of the building is not less than the total lateral force, V, determined from the requirements of the Basic Design Manual, paragraph 3-3(D), formula 3-1 (i.e., V = ZIKCSW). Complete the design in accordance with the provisions of the Basic Design Manual.

(b) Single-level design with minimum story shear requirements. Design the structure to resist EQ-I as prescribed in paragraph 4-3. However, the net story shears at each story will be at least 50 percent greater than the story shears determined from the minimum earthquake forces of the Basic Design Manual, paragraph 3-3(D). For clarification of this requirement, refer to paragraph 5-3d(2). In this procedure, the structure need not be evaluated for EQ-II.

(c) Two-level approach. Using two levels of ground motion in accordance with the procedure described in paragraph (1) above, except that the forces resulting from the EQ-I spectral response may be reduced by 30 percent (i.e., use 70 percent of EQ-I responses), unless otherwise directed. The lateral-force-resisting structural systems of these facilities will be designed to resist the modified EQ-I as prescribed in paragraph 4-3. In general, this procedure will be used only for those buildings that may be highly unusual or irregular in the distribution of mass or stiffness or in the configuration of the framing.

e. Lateral forces on structural components and nonstructural elements of structures.

(1) *Essential buildings.* All components or systems that must remain intact or functional during and after a major earthquake shall be designed with consideration of the dynamic characteristics of both the components or systems and the structure in which they occur. The accelerations and interstory drifts that are calculated from the dynamic analysis of the structure will be used, where applicable, to design components, systems, and their anchorages. For the design criteria for nonstructural elements, refer to chapter 6.

(2) *High-risk and other buildings.* All components or systems essential to life safety, which must remain intact during and after a major earthquake, will be designed in accordance with the Basic Design Manual or the design criteria for nonstructural elements in chapter 6.

f. Dynamic analysis procedures for structures other than buildings. For design criteria for structures other than buildings, refer to chapter 7.

4-3. Elastic design provisions.

The structure will be designed to resist the forces caused by design earthquake EQ-I that has a 50-percent probability of being exceeded in 50 years, or as otherwise specified by approval authority (see para 1-1*c*), in accordance with the criteria prescribed in this paragraph.

a. Method of analysis. The total lateral design force representing earthquake effects and its distributions will be determined by a response spectrum modal analysis. This requirement does not prohibit the use of a properly substantiated time history response analysis procedure.

b. Design response spectrum. The response spectrum representing EQ-I will be determined from the methodology prescribed in chapter 3, section II or III, as applicable. The damping value will be determined from table 4-1. The requirement is that the structure will resist these forces by elastic, or *nearly* elastic, behavior. Nearly elastic behavior is defined in paragraph *e* below.

Table 4-1. Damping values for structural systems.

Structural System	Elastic-Linear	Post Yield
Structural Steel	3%	7%
Reinforced Concrete	5%	10%
Masonry Shear Walls	7%	12%
Wood	10%	15%
Dual Systems	(1)	(2)

1. Use the value of the primary, or more rigid, system. If both systems are participating significantly, a weighted value, proportionate to the relative participation of each system, may be used.

2. The value for the system with the higher damping value may be used.

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c. Modal analysis methods. For a building that is regular and essentially symmetrical in size, shape, and configuration, a two-dimensional model (a vertical plane with vertical and horizontal movement within the plane) will generally be sufficient for the modal analysis of the structure in each of its two horizontal components of motion. When a structure is unavoidably not symmetrical in plan (refer to para 1-3*a*(2) for requirements), has unavoidable discontinuities in the vertical or horizontal planes (refer to para 1-3*a*(2) for requirements), has large length-to-width ratios, has flexible horizontal diaphragms, or has other irregularities, a three-dimensional model will be required for the modal analysis.

(1) *Two-dimensional (2-D) models.* The modal analysis procedure for two-dimensional models is outlined in paragraphs (*a*) through (*i*) below. Variations of this procedure may be acceptable with proper justification and approval. Additional guidelines are included in paragraph 5-4 and design examples are illustrated in appendix F.

(a) Mathematical model. The building will be modeled as a system of masses lumped at each floor level, each mass having one degree of freedom, that of lateral displacement in the direction under consideration. The computed masses will be in conformance with the weights prescribed in the Basic Design Manual, paragraph 3-3(D)5. The stiffness of the lateral-force-resisting system will be determined by established methods in accordance with the guidelines in paragraph 5-4*b* of this manual.

(b) Mode shapes and periods of vibration. The analysis will include, for each major axis, all significant modes of vibration with a minimum of three modes for buildings with 6 or more stories. The relative significance of higher modes will be determined by the values of modal participation factors and modal spectral accelerations (see para 5-4*c*(2) for additional discussion). The natural periods and mode shapes will be computed by established methods of structural mechanics and in conformance with the mathematical model described in paragraph (*a*), above.

(c) Modal story participation factor. The story modal participation factor will be calculated for each mode using the equation 4-1:

$$PF_{xm} = \left(\frac{\sum_{i=1}^{n} \frac{W_i}{g} \phi_{im}}{\sum_{i=1}^{n} \frac{W_i}{g} \phi_{im}^2} \right) \phi_{xm} \qquad \text{(eq 4-1)}$$

where:

PF_{xm} = Modal participation factor at level x for mode m.

w_i/g = Mass assigned to level i.

ϕ_{im} = Amplitude of mode m at level i.

ϕ_{xm} = Amplitude of mode m at level x.

n = Level n.

It should be noted that some references define the "modal participation factor" as the quantity within the brackets in equation 4-1 above. Also, in some references, ϕ is normalized to 1.0 at the uppermost mass level and other references will normalize the value of $\Sigma(w/g)\phi^2$.

(d) Modal base shear participation factor. The effective modal weight (or modal base shear participation factor) will be calculated for each mode using the equation 4-2:

$$\alpha_m = \frac{\left(\sum_{i=1}^{n} \frac{W_i}{g} \phi_{im} \right)^2}{\sum_{i=1}^{n} \frac{W_i}{g} \sum_{i=1}^{n} \frac{W_i}{g} \phi_{im}^2} \qquad \text{(eq 4-2)}$$

where:

α_m = Modal base shear participation factor for mode m. ($\alpha_m = C_{bm}/S_{am}$ where C_{bm} is the modal base shear coefficient and S_{am} is the modal spectral acceleration).

(e) Modal story lateral forces. The lateral forces for mode m are calculated using the equation 4-3:

$$F_{xm} = PF_{xm} S_{am} w_x \qquad \text{(eq 4-3)}$$

where:

F_{xm} = Story lateral force at level x for mode m.

w_x = Weight at or assigned to level x.

S_{am} = Spectral acceleration for mode m from the design response spectrum prescribed in paragraph 4-3*b* (as a ratio of the acceleration of gravity, g).

(f) Modal base shear. The total lateral force corresponding to mode m is calculated using the equation 4-4:

$$V_m = \alpha_m S_{am} W \qquad \text{(eq 4-4)}$$

where:

V_m = Total lateral force for mode m.

W = Total dead load of the building and applicable portions of other loads (Basic Design Manual, para 3-3(D)5)).

(g) Modal shears and moments. Story shears and overturning moments for the building and shears and flexural moments for the structural elements will be computed for each mode separately, by linear analysis, in conformance with the story forces determined in equation 4-3.

(h) Modal deflections and drifts. Modal lateral story displacements will be calculated using the equation 4-5:

$$\delta_{xm} = PF_{xm} S_{dm} = PF_{xm} S_{am} (T_m/2\pi)^2 g \qquad \text{(eq 4-5)}$$

where:

δ_{xm} = Lateral displacement at level x for mode m.

S_{dm} = Spectral displacement for mode m calculated from the response spectrum for EQ-I.

T_m = Modal period of vibration.

The modal interstory drift in a story, Δ_{xm}, will be computed as the difference of the displacements, δ_{xm}, at the top and bottom of the story under consideration (i.e., $\Delta_{xm} = \delta_{x+1)m} - \delta_{xm}$).

(i) Combinations of modal values. The combined effects of the individual modal actions (shears, moments, axial forces, etc.) and deformations (lateral story displacements, interstory drifts, etc.) for the structure and the members will be obtained by taking the square-root-of-the-sum-of-the-squares (SRSS) of the values of all significant modes. These total values are subject to modification by other provisions of this chapter (e.g., torsional, orthogonal, see para 4-3*e*).

(2) *Three-dimensional (3-D) models.* When a 3-D analysis of a building is used or is required, some modification to the procedure outlined for 2-D models (paragraphs (1)(a) through (1)(i) above) will be necessary. These modifications will be most significant for structures with large eccentricities, for structures that do not have orthogonal axis of symmetry, and for structures where the forces are applied from a direction that is not parallel to one of the major

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axes of the building. Supplementary requirements to those for 2-D models are listed below. Guideline procedures are included in paragraph 5-4.

(a) At each floor level, there will be three degrees of freedom. The primary displacement will generally occur in the component parallel to the direction under consideration. There will also be a displacement component normal to the direction under consideration and rotation about the vertical axis of the building. When the floor diaphragm is not rigid, the horizontal flexibility will be considered.

(b) A minimum of nine modes will be required in order to include three horizontal modes in each of the principal directions and three torsional modes. The possible coupling effects of the various components of motion will also be investigated.

(c) Modal story participation factors in equation 4-1 will be adjusted for 3-D effects (refer to para 5-4*d*(2) for clarification).

(d) Modal base shear participation factors in equation 4-2 will be adjusted for 3-D effects (refer to para 5-4*d*(2) for clarification).

(e) Modal story lateral forces will have three components: primary forces in the direction under consideration, forces normal to the direction under consideration, and a torque due to rotational motion.

(f) Modal base shears will have three components consistent with (*e*) above.

(g) Modal shears and moments will be determined from three components consistent with (*e*) and (*f*) above.

(h) Modal displacements and drifts will vary within the horizontal plane of each floor level as well as along the vertical axis.

(i) The total forces and deformations for the structure and the members will be obtained by an approved method to account for a rational combination of the modal values.

d. Minimum lateral forces. The story shears and story overturning moments determined from the elastic design modal analysis will be compared to the lateral static shears and overturning moments prescribed by the Basic Design Manual. If the values obtained from the modal analysis are less than the values prescribed by the Basic Design Manual (including adjustments for load factors and stress requirements), a reevaluation of the site specification of ground motion and of the dynamic structural model will be made and a statement justifying the lower story forces will be provided in the design analysis. In lieu of a justifying statement, the forces will be proportioned upwards to conform to the base shear prescribed by the Basic Design Manual. In no case will the total lateral force at the base of the structure be less than 3 percent of the total dead load of the building, W, in zones of high seismicity and 2 percent in other areas. Zones of high seismicity include seismic zones 3 and 4 of the Basic Design Manual and areas where the effective peak accelerations are greater than 0.20 in figures 3-40 and 3-41.

e. Structural component load effects. All building components will be provided with strengths sufficient to resist the combined effects of the seismic forces prescribed herein and applicable gravity loads. The requirements of paragraph 4-2*d* state that the structure will resist the seismic forces by elastic behavior. However, in some cases, nearly elastic behavior is applicable.

(1) *Nearly elastic behavior.* Nearly elastic behavior is interpreted as allowing some structural elements to slightly exceed specified yield stresses on the condition that the elastic-linear behavior of the overall structure is not substantially altered. For a structure that has a multiplicity of structural elements that form the lateral-force-resisting system, the yielding of a small number of elements will generally not effect the overall elastic behavior of the structure if the excess load can be redistributed to other structural elements that have not exceeded their yield strengths. In lieu of a substantiated analytical procedure, this condition will be considered satisfied by allowing the following percentages of exceedance to the elastic capacity requirements of paragraph 4-3*f* (based on a linear analysis).

(a) Ductile framing systems. Ductile framing systems are defined as those systems conforming to Basic Design Manual classifications for K = 0.67 or 0.80. For these systems, a limited number of the lateral-force-resisting structural elements in the direction of the force may exceed the flexural elastic capacity requirements of paragraph 4-3*f* by a value of up to 25 percent (e.g., the load combinations of paragraph (2) below will be equal to or less than 1.25 times the elastic capacity (EC). The number of horizontal flexural elements having flexural overstresses is limited to 20 percent of the horizontal seismic-resisting elements in the direction of the force on any story. The number of vertical elements having flexural overstresses is limited to 10 percent of the vertical seismic elements on any story.

(b) Other framing systems. Framing systems conforming to Basic Design Manual

classifications for K = 1.0 may have a limited number of the lateral-force-resisting structural elements in the direction of the force that exceed the flexural elastic capacity requirements of paragraph 4-3*f* by 10 percent (e.g., the load combinations of paragraph (2) below will be equal to or less than 1.10 times the elastic capacity (EC). The number of horizontal elements having flexural overstresses at any story is limited to 20 percent and the number of vertical elements having flexural overstresses at any story is limited to 10 percent.

(c) Box systems. Lateral-force-resisting systems that have the Basic Design Manual classifications with K greater than 1.0 may not exceed the elastic capacity requirements of paragraph 4-3*f*.

(2) *Design load combinations.* The structure will have the elastic capacity (EC) to resist the effects of the design load combinations shown in equations 4-6 and 4-7 (refer to para 5-4*e*(1) for clarification):

$$EC \geq 1.2D + 1.0L + 1.0E \qquad \text{(eq 4-6)}$$

$$EC \geq 0.8D + 1.0E \qquad \text{(eq 4-7)}$$

where:

EC = Elastic capacity required to resist the loads or their effects

D = Dead load

L = Live Load

E = Earthquake

(3) *Vertical accelerations.* The vertical component of earthquake motion (i.e., up and down motion) will be considered in the design of horizontal cantilever and horizontal prestressed elements. For horizontal cantilever elements, these effects will be satisfied by designing for a net upward force of 0.2D as an additional load case. For other horizontal elements employing prestressing, these effects will be satisfied by substituting equation 4-8 for equation 4-7.

$$EC \geq 0.5D + 1.0E \qquad \text{(eq 4-8)}$$

where D represents the member forces due to the vertical dead weight and E represents those due to the horizontal earthquake forces (refer to para 5-4*e*(1) for clarification). These provisions parallel those of the Basic Design Manual, paragraph 4-4*c*(2)(*a*).

(4) *Orthogonal effects.* In general, the horizontal design earthquake forces are applied nonconcurrently in the direction of each of the main axes of the structure. However, in some cases a more severe condition may occur when the force is applied at a horizontal direction not parallel to the main axes. For some elements of a building, the effects of concurrent motion about both principal axes should be investigated. Refer to Basic Design Manual, paragraph 4-4*c*(1), for additional considerations.

(5) *Horizontal distribution of forces and torsional moments.* Forces will be distributed in proportion to the relative rigidities (Basic Design Manual, para 3-3(E)4) and a minimum torsional eccentricity of 5 percent will be applied (Basic Design Manual, para 3-3(E)5). Guidelines and alternative procedures are discussed in paragraph 5-4 of this manual.

(6) *Overturning.* Structures will be designed to resist the overturning effects in accordance with Basic Design Manual, paragraph 3-3(F). Guidelines and alternative procedures are discussed in paragraph 5-4 of this manual.

(7) *Lateral displacements and drift limits.* Structures will be designed to limit the lateral displacements and interstory drifts calculated in accordance with paragraph 4-3*c* to the following values:

(a) Drift. Lateral deflections, or drift, of a story relative to its adjacent stories will not exceed 0.005 times the story height for *essential facilities.* For high-risk and other buildings, this limit is 0.007.

(b) Building separations. All portions of structures will be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflections from the prescribed seismic action.

f. Elastic capacity criteria. The criteria for the elastic capacity (EC) provisions herein are based on yield strength capacities of the structural components. Thus, the provisions for the material strengths prescribed in the Basic Design Manual and other applicable agency manuals will be upgraded to a yield strength criteria for seismic forces in combination with applicable gravity loads.

(1) *Reinforced concrete design.* The criteria used to design reinforced concrete will be the ACI Building Code (ACI 318 without app A) as modified in the Basic Design Manual.

(2) *Structural steel design.* In lieu of a strength design criteria for structural steel, working stresses specified in agency manuals for ordinary or nonseismic construction may be increased by 70 percent (e.g.,

$$\frac{f_a}{F} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.7).$$

(3) *Reinforced masonry design.* In lieu of a strength design criteria for reinforced masonry, working stresses specified in agency manuals for ordinary or nonseismic construction may be increased by 70 percent (e.g., $f_a \leq 1.7F_a$).

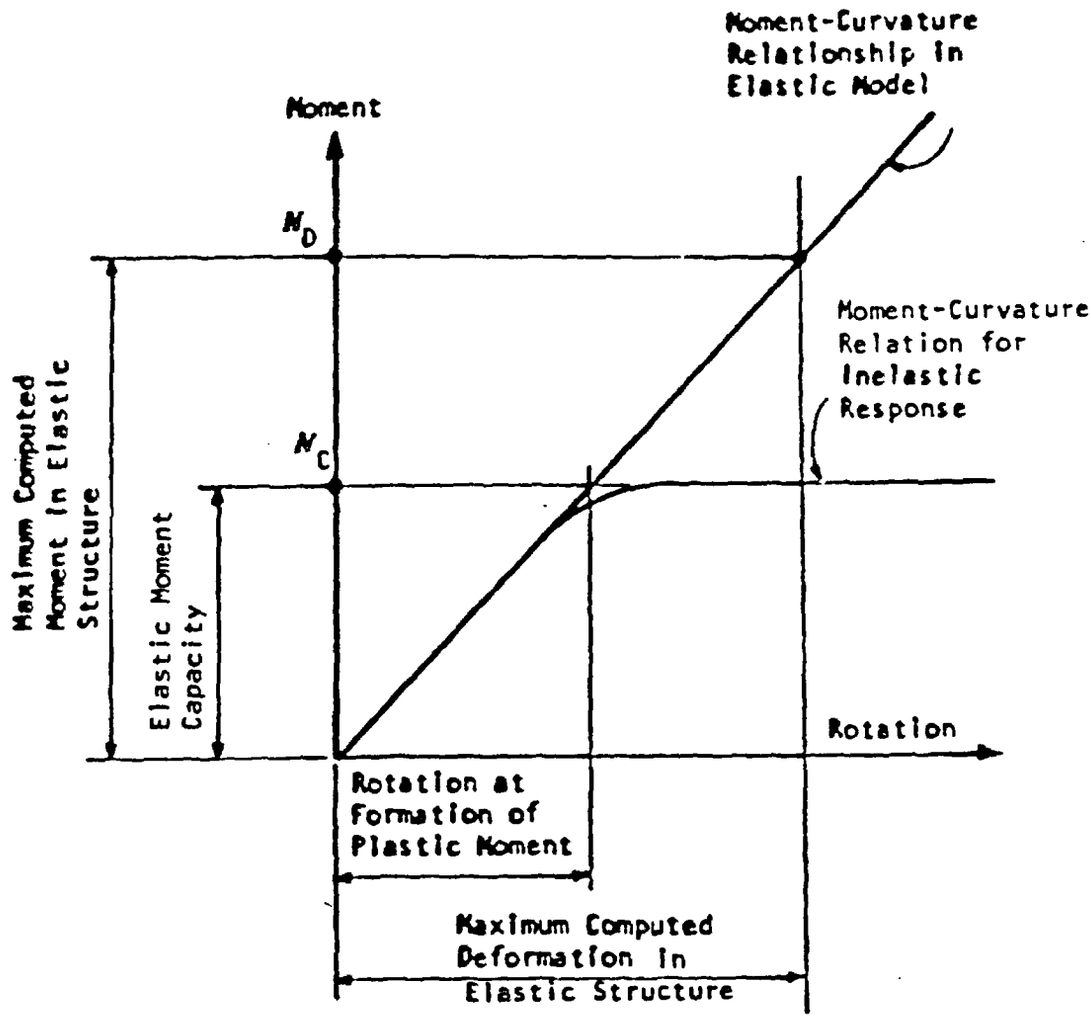
(4) *Wood design.* In lieu of a strength criteria for wood construction, working stresses specified in agency manuals for ordinary or nonseismic construction may be increased by 100 percent (e.g., f (calculated) $\leq$ 2.0 f (allowable)).

(5) *Connections.* All connections that do not develop the strength of the connecting members will have a strength reduction factor of $\phi = 0.75$. This reduction factor will be applied to the yield strength of the connection material.

4-4. Post-yield analysis provisions.

The structure conforming to the design criteria of paragraph 4-3 will be analyzed to resist the forces caused by design earthquake EQ-II in accordance with the criteria prescribed in this paragraph.

a. Method of analysis. The total lateral design forces and/or deformations representing



$$\text{Inelastic Demand Ratio} = \frac{\text{maximum computed moment in elastic model}}{\text{elastic moment capacity}} = \frac{M_D}{M_C}$$

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Figure 4-1. Definition of inelastic demand ratios for flexural members.

earthquake effects and their distribution will be determined by a response spectrum modal analysis procedure. Two acceptable procedures are presented: Method 1, an elastic analysis procedure that evaluates overstresses of individual elements (para *c* below); and Method 2, an approximate inelastic analysis procedure (para *d* below). Either of the two acceptable procedures may be used; however, these requirements do not prohibit the use of other properly substantiated inelastic response spectrum methods or inelastic time-history procedures.

b. Design response spectrum. The response spectrum representing EQ-II will be determined

DUCTILITY CHECK OF STEEL COLUMNS

1. At a braced location:

$$\frac{M_x}{M_{pcx}} + \frac{M_y}{M_{pcy}} \leq \mu$$

2. Stability between braced points:

$$\frac{C_{mx} M_x}{M_{ucx}} + \frac{C_{my} M_y}{M_{ucy}} \leq \mu$$

where:

P, M_x, and M_y = axial load and moments from first order elastic analysis

$$M_{pcx} = 1.18\, M_{px} \left[1 - (P/P_y)\right]$$

$$M_{pcy} = 1.19\, M_{py} \left[1 - (P/P_y)^2\right]$$

$$M_{ucx} = M_{ux} \left[1 - (P/P_{cr})\right] \left[1 - (P/P_{ex})\right]$$

$$M_{ucy} = M_{py} \left[1 - (P/P_{cr})\right] \left[1 - (P/P_{ey})\right]$$

M_{px}, M_{py} = plastic moment capacities

$$M_{ux} = M_{px} \left[1.07 - \frac{L/r_y \sqrt{F_y}}{3160}\right] \leq M_{px}$$

P_{ex}, P_{ey} = Euler buckling loads for x and y axes

$$P_{cr} = 1.7\, AF_a \left(P/P_{cr} \leq 0.5\right)$$

C_{mx}, C_{my} = $0.6 - 0.4\,(M_1/M_2) \geq 0.4$

μ = allowable ductility (inelastic demand ratio)

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Figure 4-2. Ductility check of steel columns.

DUCTILITY CHECK FOR CONCRETE COLUMNS

Compression:

$$\frac{M_x}{M_{ux}} \frac{1-\beta}{\beta} + \frac{M_y}{M_{uy}} \leq \mu$$

$$\frac{M_y}{M_{uy}} \frac{1-\beta}{\beta} + \frac{M_x}{M_{ux}} \leq \mu$$

Tension:

$$\frac{M_x}{M_{mx}} \frac{1-\beta}{\beta} + \frac{T}{T_u} \leq \mu$$

$$\frac{M_y}{M_{my}} \frac{1-\beta}{\beta} + \frac{M_x}{M_{mx}} + \frac{T}{T_u} \leq \mu$$

$$\frac{T}{T_u} < 0.5$$

where:

M_x, M_y, and T = Moments and net axial tension from elastic analysis

M_{ux} and M_{uy} = Uniaxial ultimate moment capacities from interaction diagrams

M_{mx} and M_{my} = Uniaxial ultimate moment capacities in the absence of axial load

T_u = Ultimate tensile capacity of vertical reinforcement = $\Sigma A_s F_y$

β = Coefficient from PCA Advanced Engineering Bulletin No. 20

μ = Allowable ductility (inelastic demand ratio)

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Figure 4-3. Ductility check for concrete columns.

from the methodology prescribed in chapter 3 for the earthquake ground motion that has a 10 percent probability of being exceeded in 100 years or as otherwise specified by approval authority (see para 1–1*c*). The damping values will be determined from table 4–1.

c. Method 1. Elastic analysis procedure. The structure that was designed in accordance with the criteria prescribed in the elastic design provisions of paragraph 4–3 will be reanalyzed to determine its capacity to perform to the demands of the larger earthquake represented by EQ–II. An elastic analysis procedure that evaluates overstresses of individual elements is outlined below. Guidelines for this procedure are presented in chapter 5, paragraph 5–5. Design examples are illustrated in appendix E.

(1) Perform a modal analysis of the structure (para 4–3*c*) using the appropriate EQ–II response spectrum. The stiffness of the lateral-force-resisting system and the computed periods and mode shapes will be established in accordance with the guidelines in paragraph 5–5.

(2) Calculate the forces on all of the structural elements. Load combinations are presented in paragraph *d* below. These forces will be defined as the *demand* forces and denoted with subscript D (e.g., M_D, V_D, F_D).

(3) Calculate the yield or plastic capacities of all the structural elements in the same force units used in paragraph (2) above. These forces will be defined as the *capacity* forces and denoted with the subscript C (e.g., M_C, V_C, F_C).

(4) Calculate the ratio of the *demand* forces to the *capacity* forces of all the structural elements. These ratios will be defined as the *inelastic demand ratios.* A graphical illustration for flexural members is shown in figure 4–1. A method determining the *inelastic demand ratios* for steel and reinforced concrete columns, by means of ductility ratios, is shown in figures 4–2 and 4–3. The equations in these figures were adapted from the general interaction equations for steel and concrete.

(5) Review the *inelastic demand ratios* for uniformity, symmetry, mechanisms, and relative values. Compare value to limits set forth in table 4–2. If any of the following conditions exist, the structure must be analyzed in accordance with Method 2 (para *d* below) or the deficiencies must be corrected by a redesign of the critical elements.

(a) Exceeding the inelastic demand ratios of table 4–2.

(b) Unsymmetrical yielding, on a horizontal plane, that will decrease the torsional resistance.

(c) Hinging of columns at a single story level that will cause a mechanism.

(d) Discontinuity in vertical elements that can cause instability or fracture.

(e) Unusual distributions of inelastic demand ratios.

(6) Engineering judgment is required for the structural evaluation of the post-yield analysis. If the review of the inelastic demand ratios satisfies the requirements of paragraph (5) above, it may be assumed that the inelastic drift is adequately approximated by the elastic analysis. Limits for inelastic deformation are governed by paragraph 4–4*e*. Guidelines are provided in paragraph 5–5.

d. Method 2: Capacity spectrum method. A step-by-step approach is used to approximate the inelastic capacity of the structure. This capacity is compared by means of a graphical procedure to the demands of the EQ–II response spectrum. Guidelines for this procedure are presented in paragraph 5–5. Design examples are presented in appendix E. A general outline of the procedure follows:

(1) By use of a modal analysis, determine the level of excitation that causes first major yielding of the structure (see paragraph *e* below for load combinations).

(2) Revise the stiffness or resistance characteristics of all structural elements that are within 10 percent of their yield capacities to represent a plastic hinge.

(3) Apply additional lateral forces to the structure, by means of a modal analysis, until an additional group of structural elements reaches their yield capacities.

(4) Repeat the above until the combined results reach an ultimate limit (e.g., a mechanism, instability, or excessive distortions) (see para *e* below for evaluation criteria).

(5) Convert the results into a *capacity* curve based on the periods and spectral accelerations for the fundamental mode of vibration.

(6) Graphically compare the *demand* of the EQ–II response spectrum to the *capacity* of the structure.

(7) Approximate the lateral deformations and compare to the drift limits of paragraph *e* below.

e. Evaluation criteria. The structure will be evaluated for its ability to resist the combined effects of the seismic forces prescribed herein and the applicable gravity loads within the prescribed lateral distortion limits.

(1) *Load combinations.* The demands on the structure will be equal to the combined effects of the dead (D), live (L), and seismic (E)

Table 4-2. Inelastic demand ratios.

Building System	Element	Essential	High Risk	Others
Steel DMRSF	Beams	2.0	2.5	3.0
	Columns*	1.25	1.5	1.75
Braced Frames	Beams	1.5	1.75	2.0
	Columns*	1.25	1.5	1.75
	Diag. Braces**	1.25	1.5	1.5
	K-Braces***	1.0	1.25	1.25
	Connections	1.0	1.25	1.25
Concrete DMRSF	Beams	2.0	2.5	3.0
	Columns*	1.25	1.5	1.75
Concrete Walls	Shear	1.25	1.5	1.75
	Flexure	2.0	2.5	3.0
Masonry Walls	Shear	1.1	1.25	1.5
	Flexure	1.5	1.75	2.0
Wood	Trusses	1.5	1.75	2.0
	Columns*	1.25	1.5	1.75
	Shear Walls	2.0	2.50	3.0
	Connections (other than nails)	1.25	1.50	2.0

*In no case will axial loads exceed the elastic buckling capacity.

**Full panel diagonal braces with equal number acting in tension and compression for applied lateral loads.

***K-bracing and other concentric bracing systems that depend on compression diagonal to provide vertical reaction for tension diagonal.

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loads shown in equations 4-9 and 4-10:

$$\text{Demand} = D + L^* + E \qquad \text{(eq 4-9)}$$

$$\text{Demand} = D + E \qquad \text{(eq 4-10)}$$

where the live load (L*) is equal to a realistic estimate of the actual live load. The value of L* may be as low as 25 percent of the design live load (L).

(2) *Lateral displacements and drift limits.*

(a) Drifts. Interstory drifts will not exceed 0.010 times the story height for *essential faciities.* For high-risk buildings and all other buildings, the limit is 0.015.

(b) Building separations. Under the conditions of these requirements, some contact between buildings is acceptable if it can be shown that the effects of pounding will not cause loss of function, instability of the affected portion of the structure, or hazard to life-safety. For example, if all the floors of adjacent buildings are in vertical alignment with each other, then the pounding associated with the extreme conditions of EQ-II might cause only some minor local damage to the material in contact. However, if the floor of one building was in alignment with mid-height of columns in the adjacent building, pounding could cause column instability due to buckling and P-delta effects. If some contact is acceptable for EQ-II, the minimum separation between buildings will be governed by the requirements for EQ-I as prescribed in paragraph 4-3e(7)(*b*). If contact is to be avoided

for EQ–II, the minimum separation between buildings will be governed by the combined maximum displacements of the adjacent buildings due to the seismic actions of EQ–II. The maximum story displacements, at respective locations, may be combined by the square-root-of-the-sum-of-the-squares to determine the minimum separation.

(c) P–delta effects. The secondary effects of the lateral displacements (delta) combined with the gravity forces (P) will be investigated.

(3) Structural materials and details. Structural elements and connections will conform to the requirements of the Basic Design Manual and will be evaluated for their ability to sustain the implied ductility demands of the post-yield analysis procedures.

CHAPTER 5
STRUCTURAL DESIGN PROCEDURE

5-1. Introduction.

This chapter describes general procedures for the design and analysis of buildings to resist the earthquake lateral forces specified in chapter 4, Criteria for Structural Analysis. Guidelines are provided for a dynamic analysis approach to seismic design of buildings. Guidelines for conventional static force procedures are provided in the Basic Design Manual, chapter 4.

5-2. Preliminary design considerations.

a. Design response spectra. Before proceeding with the design of a building by means of a dynamic analysis approach, geotechnical data will be required to determine the design ground motion and foundation design criteria. The methodology for specifying the ground motion and site-specific response spectra for a particular site is prescribed in chapter 3, Specification of Ground Motion. Unless otherwise specified by approval authority (para 1-1c), the following criteria will apply:

(1) *EQ-I response spectrum.* The response spectrum representing EQ-I has a 50-percent probability of being exceeded in 50 years.

(2) *EQ-II response spectrum.* The response spectrum representing EQ-II has a 10 percent probability of being exceeded in 100 years.

(3) *Damping.* Damping values will be as indicated in table 4-1.

b. Selection of structural system. The possibility of structural damage and collapse can be minimized by effective structural planning. For general guidelines to the selection of the structural system, refer to the Basic Design Manual, paragraph 2-8. The objectives of effective structural planning are to maintain symmetry, minimize building torsion, provide direct vertical paths for lateral forces, and to provide proper foundations. A continuous load path, or paths, with adequate strength and stiffness that will transfer all forces from the point of application to the final point of resistance must be provided. The foundations must be designed to accommodate the forces developed or the moments imparted to the building by the design ground motions. Additional discussions on techniques of seismic design, path of forces, and design of foundations are covered by the Basic Design Manual, paragraphs 2-9, 4-4*d*, and 4-8.

c. Capacities of buildings to resist demands of earthquakes. The ability of structures to resist the excessive accelerations and deformations of severe earthquakes is not directly proportional to the equivalent design seismic forces or to the amplitudes of the peak ground accelerations of earthquakes. The design strength of a structure is governed by a combination of lateral-force criteria (e.g., wind and earthquake) and gravity load criteria (e.g., dead and live loads). Some of the excess capacity built into the gravity load design will be available to resist lateral forces. In addition, if the structure has ductility and/or redundancy, it will respond to excessive lateral forces in an inelastic manner thay may result in demands that are less severe than the demands applied to a fully elastic structure. This can be explained by the decrease in stiffness due to inelastic action, which lengthens the effective period of vibration, and by the increase in energy absorption and the reduction in response amplification due to inelastic action. These effects are represented by a longer structural period together with a larger value for effective damping. These relationships are illustrated in figures 5-1, 5-2, and 5-3.

d. Foundation capacities to resist demands of earthquakes. The geotechnical and/or soils foundation consultant will establish criteria based on ultimate capacities of the soils to resist the effects of short-term seismic loading conditions in combination with the long-term gravity loading. For load combinations with EQ-I, the soil capacities must be sufficient to provide resistance essentially within the elastic limits of the soil. A factor of safety of 2 on the ultimate capacity is recommended. For load combinations with EQ-II, the soil capacity must be sufficient to prevent sudden failure of the soil. Some minor differential movement due to soil deformation is acceptable under the conditions of EQ-II.

5-3. General design procedures.

The scope of this chapter covers design procedures for three general classifications of structures: essential facilities; high-risk; and all other buildings. A general flow chart is shown in table 5-1. Outlines of the general procedures for each of the three classifications are presented in tables 5-1*a*, 5-1*b*, and 5-1*c*, respectively.

a. Initial trial design. In many cases, a building designed in accordance with the static force procedure of the Basic Design Manual will satisfy the requirements of the dynamic analysis procedure of this manual with little or no mod-

Gravity/Seismic-Load Relationships. Because of the relationships between gravity loads (dead load (DL) and live load (LL)) and lateral forces (seismic loads), the stresses in the structural elements are not directly proportional to the seismic forces. For example, if the lateral forces are tripled, the combined stresses in the structural elements will not necessarily triple because the dead load and live load stresses will remain essentially constant.

To illustrate these relationships, sample calculations, which assume a beam with negative bending moments at the supports of -100 k-ft, are shown below:

1. Negative seismic bending moments at end of beam.

   a. Design Bending Moments:

DL + LL	=	-100 k-ft
Seismic	=	- 50 k-ft
Total Design Moment	=	-150 k-ft

   b. Triple Seismic Forces:

DL + LL	=	-100 k-ft
Seismic	=	-150 k-ft
Total Beam Bending Moment	=	-250 k-ft

   c. Ratio of Triple Seismic Forces to Design Forces (b ÷ a):
   250 ÷ 150 = 1.67 < 3.00

2. Positive seismic bending moments at end of beam.

   a. Design Bending Moments (consider DL moment only):

0.9 DL = 0.9 (-70)	=	-63 k-ft
Seismic	=	50 k-ft
Net Moment (no load reversal)	=	-13 k-ft

   b. Triple Seismic Forces:

0.9 DL	=	- 63 k-ft
Seismic	=	150 k-ft
Net Moment (Reverses to positive bending moment)	=	87 k-ft

   c. Ratio of Triple Seismic Forces to Design Forces:
   Case a: No positive bending moment
   Case b: 87 k-ft positive bending moment
   87 ÷ 0 = ∞ (infinity)

3. Axial forces on a column.

   a. Design Axial Forces:
   0.9 DL > Seismic Axial Force
   Therefore, no tension in column

   b. Triple Seismic Forces:
   0.9 DL < Seismic Axial Force
   Therefore, there is tension in column

   c. Ratio of Triple Seismic Forces to Design Forces, similar to Sample 2, is equal to infinity.

Reprinted from "An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance," ATC-10, U.S. Geological Survey, 1982.

Figure 5-1. Gravity/seismic load relationships.

Dynamic Structural Characteristics. Because of the relationships between dynamic characteristics of structures and the dynamic properties of seismic ground motion, the effective forces applied to the structure are not directly proportional to the peak ground acceleration of the earthquake. The periods of vibration of a building, as well as the effective damping of the structure will vary with the amplitude of motion. For example, a building will respond at a certain period and damping value for a moderate earthquake in an elastic manner. For an earthquake two times larger (e.g., response spectra with twice the spectral accelerations), some structural elements may exceed their elastic limits, the period of vibration will be slightly longer and the damping will increase; thus, the spectral acceleration for the larger earthquake will be less than twice the value of the moderate earthquake. These relationships are illustrated in sample response spectra shown in Figure 1 and are summarized below:

1. The sample building responds elastically at Point A (S_a = 0.8 g), for earthquake E-Q-1 at 5% damping (Peak ground acceleration, A_G, is 0.3 g).

2. If the building remained elastic for A_G equal 0.6 g, the building would respond at Point B (S_a = 1.6 g) for earthquake E-Q-2. But the building does not remain elastic because some structural elements yield.

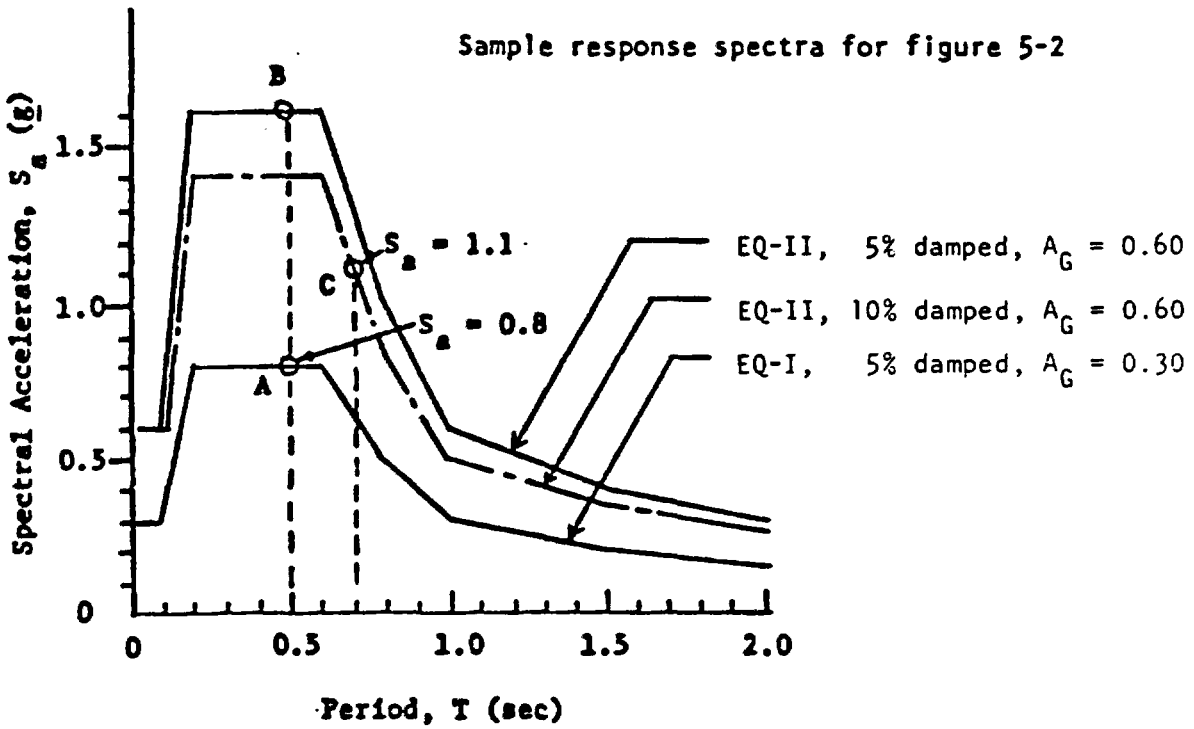
3. The fundamental building period shifts from 0.5 seconds to an effective value of 0.7 seconds due to stiffness degradation. Due to inelastic response and energy absorption, the effective damping increases from 5% to 10%. Thus, the sample building has a peak response at Point C (S_a = 1.1) for earthquake E-Q-2.

4. Therefore, the peak response of the building is 40% greater (1.1 g vs. 0.8 g) for an earthquake ground acceleration twice as large (0.6 g vs. 0.3 g).

Structures with degrading stiffnesses are extremely sensitive to the time factor in earthquake behavior. Reduction in stiffness occurs in reinforced concrete when cracks, which open during an inelastic loading cycle, do not close on the reverse cycle due to elongation of the tension steel. This reduces the effective cross section and the corresponding stiffness. As a result, the fundamental period of vibration will tend to lengthen and the damping will tend to increase, which will increase dissipation of the seismic input. If the period elongation and the damping increase can reduce the seismic input at a faster rate than the reduction in stiffness, the structure will survive. It will simply readjust itself so that it is oscillating in an elastic manner about a new equilibrium position having a reduced stiffness and an increased damping. If geometrical effects of the vertical axial loads also contribute to the reduction in lateral stiffness, however, the stiffness may reduce faster than the seismic input. In this case, structural failure may result. In either case, the duration of strong ground shaking is the critical factor.

Reprinted from "An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance," ATC-10, U.S. Geological Survey, 1982.

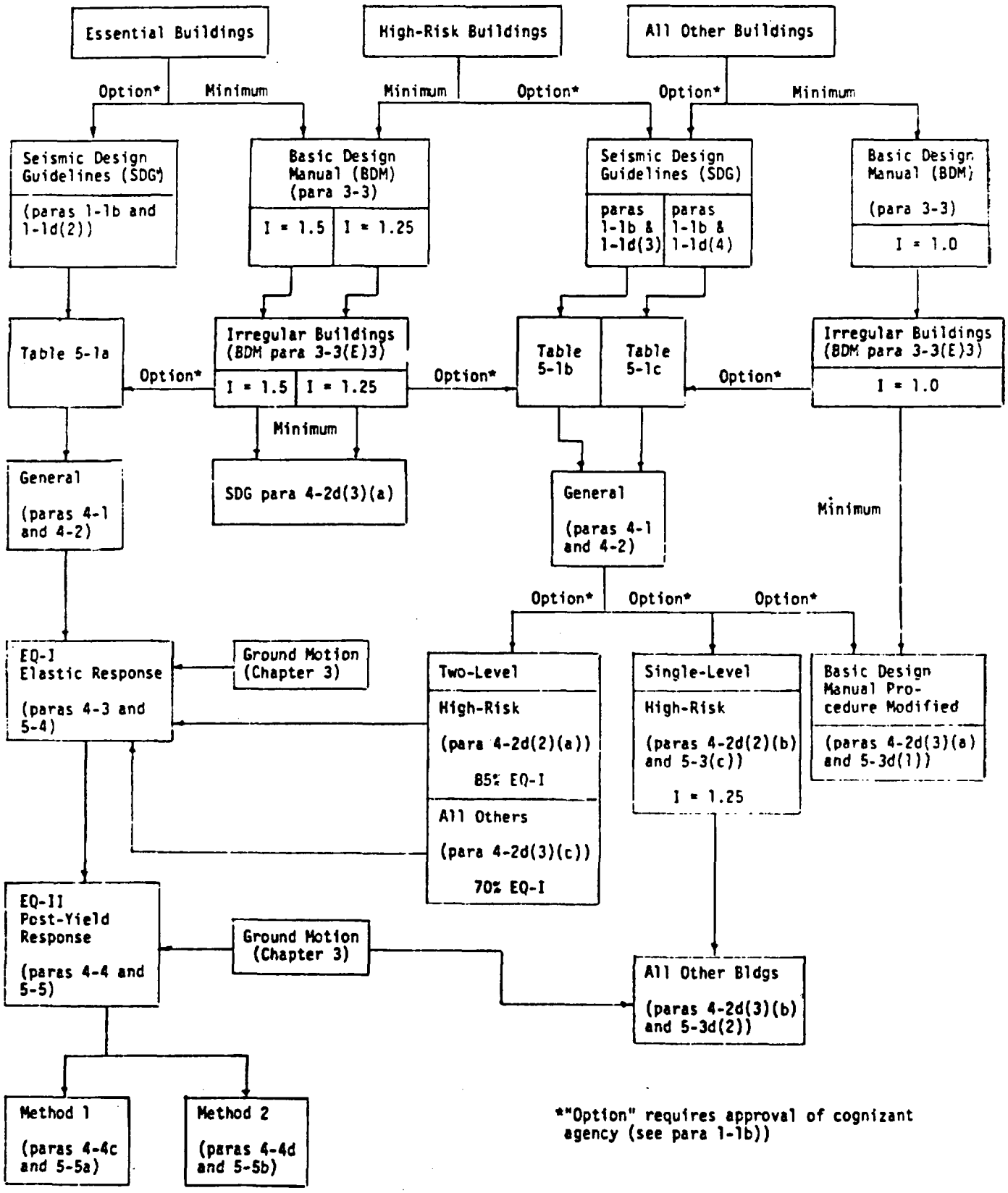
Figure 5-2. Dynamic structural characteristics.



Reprinted from "An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance," ATC-10, U.S. Geological Survey, 1982.

Figure 5-3. Nonproportional relationship between peak ground acceleration and spectral acceleration.

Table 5-1. Seismic design procedures.



*"Option" requires approval of cognizant agency (see para 1-1b))

Note: All paragraph references are to this document unless indicated as "BDM" for Basic Design Manual

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Table 5–1a. Seismic design of essential facilities.

	Requirements	Procedure
Classification of building	4-1a	
General requirements	4-2a	
Dynamic nalysis procedure	4-2d(1)	5-3b
EQ-I		
Select response spectrum	4-3b	3-6,3-8,5-2a
Select structural system	4-2a(1)	5-2b
Initial trial design		5-3a
Modal analysis	4-3c	5-4a,b,c,d
Minimum lateral forces	4-3d	5-4j
Drift limits	4-3e(7)	5-4f
Load combinations	4-3e(2),(3)	5-4e(1)
Structural components	4-3e(1),4-3f	5-4e
Orthogonal, torsion, overturning	4-3e(4),(5),(6)	5-4h,i
Foundations	4-2a(1)	5-2d,5-4h
Nonstructural	4-2e,6-2	6-3, 5-4g
EQ-II		
Select response spectrum	4-4b	3-6,3-8,5-2a(2)
Analysis procedures:		
Method 1	4-4c	5-5a
Method 2	4-4d	5-5b
Load combinations	4-4e(1)	5-5a(3)
Drift limitations and P-δ effects	4-4e(2)	5-4f,5-5b(2)(h), 5-5c,5-5d
Structural components	4-4e(3)	5-5a(4),5-5b(2)
Foundations	4-2a(1)	5-2d

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Table 5-1b. Seismic design of high-risk buildings.

	Requirements	Procedure
Classification of building	4-1b	
General requirements	4-2a	
Dynamic analysis procedure	4-2d(2)	5-3
Two Level Approach	4-2d(2)(a)	

Same as essential facilities (Table 5-1a) except the following:

EQ-I	
Response spectrur	85% of EQ-I, 4-2d(2)(a)
Drift limits	Increase 40%, 4-3e(7)
EQ-II	
Drift limits	Increase 50%, 4-4e(2)
Inelastic demand ratio	High-risk column of table 4-2

Single Level Design

Same as Basic Design Manual Procedure with modified seismic force distribution and Single level design for EQ-I with minimum story shear requirements for other buildings (Table 5-1c). Minimum lateral forces governed by Basic Design Manual will be 25% higher because the I-coefficient equals 1.25.

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Table 5-1c. Seismic design for other buildings.

	Requirements	Procedure
Classification of building	4-1c	
General requirements	4-2a	
Dynamic analysis procedure	4-2d(3)	
Basic Design Manual Procedure with modified seismic force distribution		
General	4-2d(3)(a)	5-3d
Response spectrum EQ-I	4-3b	
Select structural system	Basic Design Manual	
Initial design	Basic Design Manual	
Modal analysis	4-3c	5-3d(1)
Minimum lateral force	Basic Design Manual	
Normalize modal analysis	4-2d(3)(a)	5-3d(1)
Final design	Basic Design Manual	
Single level design for EQ-I with minimum story shear requirements		
General	4-2d(3)(b)	5-3d(2)
Response spectrum EQ-I	4-3b	
Select structural system	Basic Design Manual	
Initial trial design	Basic Design Manual	5-3a
Modal analysis	4-3c	
Minimum lateral force	4-2d(3)(b)	5-3d(2)
Drift limits	4-3e(7)	
Load combinations	4-3e(2)(3)	
Structural components	4-3e(1),4-3f	
Orthogonal, torsion, overturning	4-3e(4),(5),(6)	
Foundations	4-2a(1)	5-2d,5-4h
Two level approach		
General	4-2d(3)(c)	5-3d(3)
Same as essential facilities except for the following:		
EQ-I		
Response spectrum	70% of EQ-I	
	4-2d(3)	5-3d(3)(a)
Drift limits	Increased 40%	
	4-3e(7)	5-3(d)(3)(b)
EQ-II		
Response spectrum	4-4b	5-2a(2)
Drift limits	Increased 50%	5-4f,5-5b(2)(h),
	4-4e(2)	5-5c,5-5d
Inelastic demand ratio	Table 4-2	5-5a(4)

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ifications. The primary purpose of the procedures of this manual is to provide a more rational approach to the fulfillment of the intent of the Basic Design Manual. The initial selection of trial structural member sizes can be made in a manner similar to that of conventional static design procedures, as outlined in the Basic Design Manual. Following is a suggested procedure for the initial design. An alternate procedure is outlined in paragraph (2) below.

(1) Code comparison concept.

(a) Compare the EQ-I design spectrum with the curve representing the static base shear coefficients ZICS. For example, the EQ-I spectrum may be similar to the 5-percent curve in figure 2-8. The ZICS curve may be similar to the T_s = 1.0 curve in Basic Design Manual figure 4-3, with the C × S values multiplied by 1.5 to account for Z = 1.0 and I = 1.5. An example of these two curves is shown in figure 5-4.

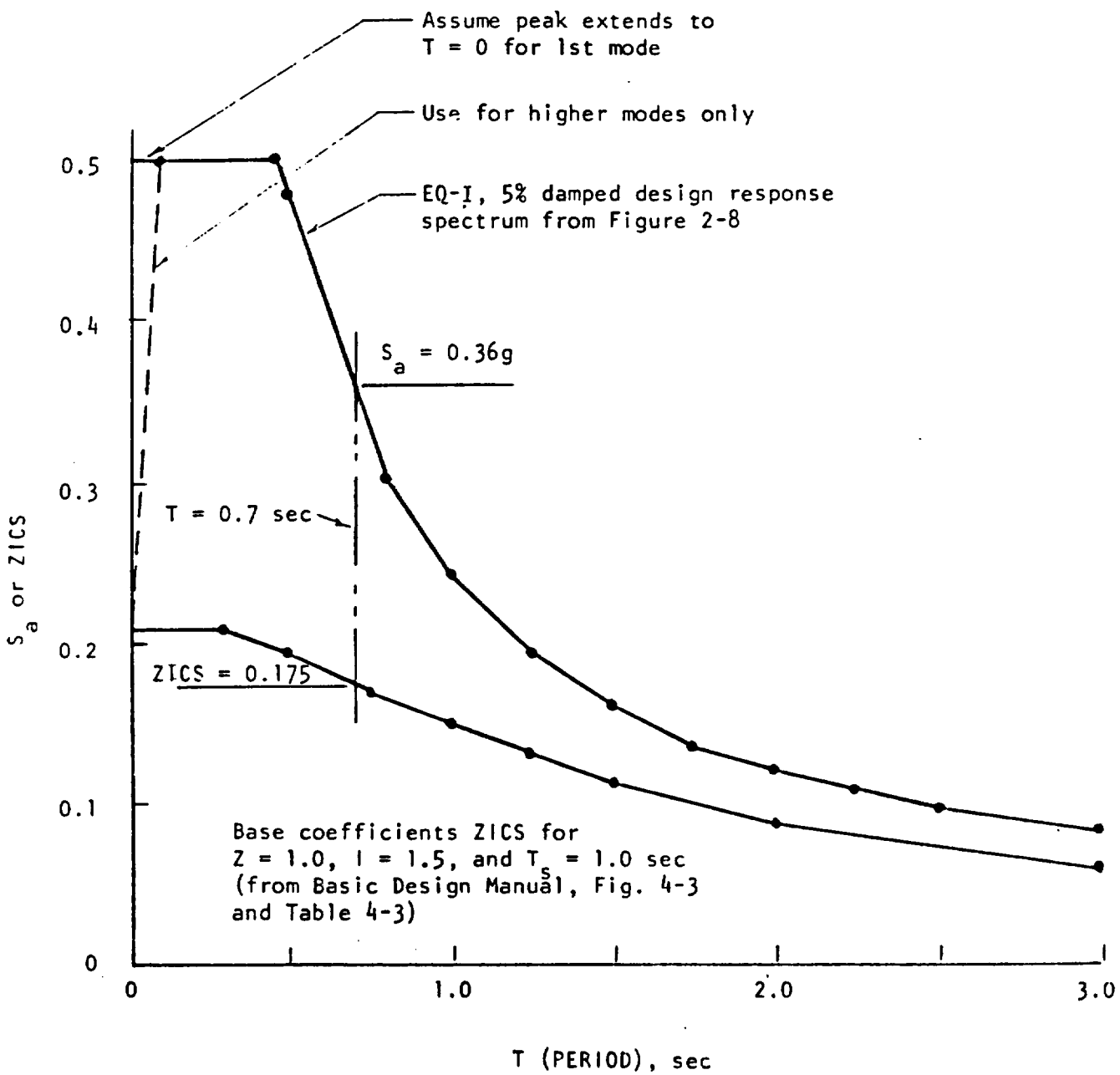

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Figure 5-4. Sample EQ-I spectrum and ZICS curve.

(b) Estimate the period of the fundamental mode of vibration of the structure by methods described in the Basic Design Manual. For example, the period of a 7-story frame structure may be estimated at 0.IN = 0.7 seconds, as illustrated in figure 5–4.

(c) Compare value of S_a of EQ–I with ZICS for the estimated building period.

1. If S_a is roughly 2 times ZICS or less, the static design procedure will probably result in a reasonable initial design. The factor of 2 is based on a combination of load factors, participation factors, and underestimation of the building period.

2. If S_a is substantially greater than 2 times ZICS (e.g., 3 to 4 times), the initial static design should be based on a porportionately higher value of ZICS.

(2) An alternate procedure is to estimate a yield level base shear coefficient directly from the EQ–I spectrum.

(a) Estimate the fundamental period of vibration.

(b) Determine the value of S_a from the EQ–I response spectrum.

(c) Estimate the fundamental base shear participation factor, α (para 4–3*c*(1)(d)), from the following:

5 stories: $\alpha = 0.80$
4 stories: $\alpha = 0.83$
3 stories: $\alpha = 0.86$
2 stories: $\alpha = 0.90$
1 story: $\alpha = 1.00$

(d) Estimate the base shear coefficient by multiplying S_a by α.

(e) Use the base shear coefficient to estimate lateral forces on the building in the same manner used in the static design procedure. Use these forces initially to size the structural members; however, the capacities will be on the basis of yield strength in lieu of allowable stresses.

(f) If S_a is not significantly greater than ZICS (e.g., 50 percent greater), refer to paragraphs 4–3*d* and 5–4*j* for minimum lateral force requirements.

b. Dynamic analysis procedure for critical and essential buildings. Critical and essential facilities will be designed to resist two levels of earthquake motion as prescribed in paragraph 4–2*d*(1). The procedure is described in paragraphs 5–4 and 5–5.

c. Dynamic analysis procedure for high-risk buildings. High-risk buildings will be designed in accordance with either a two-level approach or a single-level design, as prescribed in paragraph 4–2*d*(2). The choice will generally depend on the seismic severity of the site, type of building, criteria established for other buildings at or near the site, and the decision of the approval authority. For example, the building may be part of a large hospital complex that has essential facilities as well as high-risk buildings. The designer will have site ground motion specification data available and will have had to develop dynamic two-level approach procedures for the essential facilities. Therefore, the premium for designing the high-risk building in accordance with the two-level approach may be insignificant. In another example, the building may have unavoidable irregularities that generate concern about the ability of the structure to satisfactorily sustain a major earthquake without serious damage. Thus, a two-level approach may be justifiable. In a third case, the building may be the only building at a site where ground motion specification data are not available and where no other special conditions exist that would justify the additional effort of a two-level approach. Therefore, a single-level design procedure is adequate.

(1) *Two–level approach.* The procedure is the same as used for essential structures with the following exceptions:

(a) The EQ–I response spectrum is reduced by 15 percent (para 4–2*d*(2)(*a*)). The effect of this reduction is that the structure will remain elastic for ground motion less than that specified by EQ–I or, conversely, that some damage will be accepted for the EQ–I ground motion.

(b) The drift limits for EQ–I (0.007) and EQ–II (0.015) are less severe (paras 4–3*e*(7) and 4–4*e*(2)).

(c) The limits on inelastic demand ratios are less severe (table 4–2).

(2) *Single-level design.* The procedures are the same as used for all other buildings in paragraphs 4–2*d*(3)(*a*) and (*b*) with the exception that the minimum values will be calculated on the basis of I = 1.25. The procedures are described in paragraphs *d*(1) and *d*(2) below.

d. Dynamic analysis procedure for all other buildings. Three alternative procedures are prescribed in paragraph 4–2*d*(3): Basic Design Manual criteria with modified seismic force distribution; single-level design with minimum story shear requirements; and two-level approach. The choice will depend on the data available and on particular requirements of the facility. Paragraphs 4–2*d*(3) (*a*) and (*b*) are both single-level design procedures. These procedures will generally be sufficient for most buildings. Paragraph 4–2*d*(3)(*c*) is a two-level approach. This procedure may be required by the approval authority for buildings that have unavoidable highly

irregular configurations or other unusual conditions.

(1) Basic Design Manual criteria with modified seismic force distribution (para 4-2*d*(3)(*a*)). This procedure uses the modal analysis method to determine the distribution of lateral forces along the height of the structure in liew of the distribution determined from Basic Design Manual equations 3-6 and 3-7. For buildings with large differences in lateral resistance or stiffness between adjacent stories, the differences between the two methods can be significant. The modal analysis procedure requires a response spectrum. The EQ-I response spectrum is prescribed. However, if data are not available for EQ-I, a standardized shape for a response spectrum may be substituted (e.g., an ATC 3-06 spectrum). The amplitude of the peak ground acceleration is not significant because the results are later normalized to equal the base shear determined in the Basic Design Manual. Therefore, this procedure has the advantage of not requiring site specific earthquake data. A summary of the procedure follows:

(a) Determine the story shears, story overturning moments, story accelerations, story displacements, and interstory drifts by means of a response spectrum modal analysis in accordance with paragraph 4-3. This includes the total lateral force at the base, $V_T = \sqrt{\Sigma V_m^2}$. If data to develop the EQ-I response spectrum are not available, the equations in paragraph 3-8*c* may be used to determine values for S_a. Any single value may be used for A_a and A_v (e.g., $A_a = A_v = 0.20$), because the base shear normalization process prescribed in paragraph *(d)* below will equalize the results. The soil profile coefficient, S_i, will be determined from table 3-6 in conformance with the decriptions in table 3-5.

(b) Determine the total lateral force, V = ZIKCSW, in accordance with the Basic Design Manual.

(c) Calculate the ratio, R_v, of the Basic Design Manual base shear to the modal analysis base shear:

$$R_v = ZIKCSW/\sqrt{\Sigma V_m^2}. \qquad \text{(eq 5-1)}$$

(d) Multiply all the values in paragraph *(a)*, above, by R_v.

(e) Use the resulting story shears and overturning moments to design the building in accordance with the provisions of the Basic Design Manual.

(f) Use the story accelerations to compare with the coefficients ZIC_p of Basic Design Manual equation 3-8. The larger of the two values will be used for the design of elements of structures.

(g) Use the interstory drifts to determine conformance with the Basic Design Manual drift provisions.

(2) Single-level design with minimum story shear requirements. This procedure provides for a single-level modal analysis of the structure to withstand the actions of EQ-I in conformance with paragraph 4-3. However, a lower limit equal to 1.5 times the Basic Design Manual is specified. The 1.5 value is used to account for the differences between working stress or load factor criteria used in the Basic Design Manual and the yield strength criteria used for EQ-I. This procedure can result in significantly larger forces than the procedure described in paragraph (1) above, if the site specific earthquake, EQ-I, so indicates. In some cases, the analysis for EQ-I may result in lower force levels than those obtained from paragraph (1) above. However, the lower limit of 1.5 times the Basic Design Manual will generally keep the capacity of the resulting structure from being less than that of the structure designed in accordance with paragraph (1) above. A summary of the procedure follows:

(a) Complete the modal analysis prescribed in paragraph 4-3 for EQ-I. List all the combined modal story shears.

(b) Determine all the story shears as prescribed in the Basic Design Manual.

(c) If any story shear determined in paragraph *(a)* is not at least 1.5 times the corresponding story shear listed in paragraph *(b)*, increase all values determined by modal analysis proportionately to satisfy this requirement. For example, the modal analysis gives a third-story shear equal to 14 kips, and the Basic Design Manual method gives a third-story shear equal to 10 kips. The ratio is 1.4, which is less than 1.5. Therefore, multiply all values in the modal analysis by 1.07 (1.5 ÷ 1.4 = 1.07).

(d) Use the revised values to complete the design of the building in accordance with the provision of this manual to resist EQ-I. The structure need not be evaluated for EQ-II.

(3) Two-level approach. This procedure is the same as that used for essential facilities (exceptions in paras *(a)*, *(b)*, and *(c)*, below) and is substantially more complex than the procedures in paragraphs (1) and (2) above, it will only be used under special conditions, as directed by the approval authority, such as for highly irregular or unusual buildings. The discussion in paragraph 5-3*c* on the use of the two-level approach for high-risk buildings also applies. The procedure for the two-level approach

is described in paragraphs 5-4 and 5-5. Exceptions are listed below:

(a) The EQ-I response spectrum is reduced by 30 percent (para 4-2*d*(3) *(c)*). The effect of this reduction is similar to that indicated above for high-risk facilities (para 5-3*c*(1) *(a)*).

(b) The drift limits for EQ-I (0.007) and EQ-II (0.015) are less severe (para 4-3*e*(7) and 4-4*e*(2)).

(c) The limits on inelastic demand ratios are less severe (table 4-2).

5-4. Designing for EQ-I.

The structure will be designed to resist the forces of EQ-I within the elastic range of the capacity of the lateral-force-resisting system. An initial trial design is developed in accordance with paragraph 5-3*a*. The initial design is then checked for conformance to the criteria by means of a modal analysis for the EQ-I response spectrum.

a. Modal analysis procedure. Periods, mode shapes, and participation factors are required, in conjunction with the design response spectrum, to perform a dynamic analysis. The accuracy of these factors and the degree of sophistication required in the analysis is dependent on the size and complexity of the building.

(1) *Single-story building.* Unless the building is unusual or irregular in plan, the modal analysis procedure essentially becomes equivalent to a static design procedure.

(a) The period of vibration will generally be in the range of 0.1 to 0.2 seconds, thus placing it at the peak of the response spectrum for a maximum value of S_a. Note that the peak of the response spectrum is assumed to extend back to T = 0 for the fundamental mode as noted in figure 5-4. In general, even a very rigid structure with a short natural period of vibration will respond at a slightly longer period due to soil-structure interaction.

(b) For a single-story building, the base shear participation factor will be equal to unity (e.g., $\alpha = 1.0$). Therefore, the base shear coefficient will be equal to the spectral acceleration, S_a.

(c) The total lateral force on the building, for each direction of motion, will be equal to the spectral acceleration times the weight of the building ($V = S_a \times W$) in accordance with equation 4-4.

(2) *Low-rise buildings up to about 5 stories.* Unless the building is unusual or irregular in elevation or plan, the modal analysis can generally be limited to the fundamental mode of vibration. Although the use of a computer program will generally be more efficient and will generally give more accurate results, the single-mode analysis can be done by hand calculations.

(a) Estimate the fundamental period of vibration (e.g., Basic Design Manual, equations 3-3A or 3-3B), assume a straightline mode shape and calculate or estimate the story weight.

(b) Calculate the modal participation factors PF_x and α. Approximate the spectral acceleration, S_a, for the estimated period using the EQ-I response spectrum.

(c) Calculate the story forces, F_x (refer to appendix E, design example E-1, for the procedure).

(d) Calculate the deflected shape of the structure. This can be done by hand calculations (though somewhat difficult and time-consuming) or with the aid of a computer program.

(e) Use the calculated deflected shape as a new estimate for the mode shape and repeat paragraphs (*b*) and (*c*) above.

(f) If the story forces of paragraph (e) compare favorably with the original values of paragraph (*c*) (e.g., within about 10 percent), assume the deflected shape of paragraph (*d*) to be acceptable. If not, repeat paragraph (*d*) to calculate the deflected shape for the revised story forces.

(g) Calculate the period of vibration using the Basic Design Manual equation 3-3. A quicker method is by means of the following equation, using the forces and displacements calculated above:

$$T = 2\pi \sqrt{\delta_n w_n / F_n g} \qquad \text{(eq 5-2)}$$

where δ_n, w_n, and F_n are the displacement, weight, and force at the roof. This equation can be derived from equations 4-3 and 4-5.

(h) If the period of vibration calculated in paragraph (*g*) above is substantially different than the value assumed in paragraph (*a*) above, repeat paragraph (*b*) and adjust the forces and displacements in proportion to the new value for S_a.

(3) *Moderate-rise buildings from 5 to about 15 stories.* For buildings over 5 stories, some of the effects of higher modes of vibration may be significant. In lieu of a detailed analysis, the dynamic characteristics can be approximated. Table 5-2 shows the general modal relationships for a fairly uniform 7-story reinforced concrete frame building. For a 14-story building, a modal analysis could be approximated as follows:

(a) Estimate the fundamental period of vibration (e.g., Basic Design Manual, equations 3-3A or 3-3B).

(b) Approximate periods for the second through fifth modes of vibration using the ratios shown in table 5-2 (e.g., second mode period equals 0.327 time the fundamental mode period).

(c) Approximate the mode shapes by using the shapes shown in table 5-2 and interpolating for the taller structure (e.g., for the second mode, assume 1.00 for the roof and 0.550 for the 13th story. Estimate the 14th story at 0.775).

(d) The participation factors can be taken directly from table 5-2 or new values can be calculated from the mode shape by using equations 4-1 and 4-2.

(e) Determine the spectral accelerations, S_a, for each modal period from the response spectrum.

(f) Calculate story forces for each of the modes as shown in appendix E, design example E-1. The results for the 7-story building are summarized in table 5-3 and are illustrated graphically in figure 2-10.

(g) Calculate the deflected shape of the building separately for each mode of vibration. This will generally require the use of a computer. Compare the deflected shapes to the mode shapes approximated in paragraph (*c*), above. (*Note*: some computer programs will perform paragraphs (*a*) through (*g*), above, directly.) If the shapes are similar, continue with the analysis. If there are significant differences in mode shapes, a modification of paragraphs (*d*) through (*g*), above, may be required.

(h) Calculate the periods of vibration using the Basic Design Manual equation 3-3. An alternate method is to use equations 4-3 and 4-5 and solve for T_m for each mode at several story levels as follows:

$$T_m = 2\pi \sqrt{\delta_{xm} w_x / F_{xm} g} \qquad \text{(eq 5-3)}$$

Table 5-2. General modal relationships.

Mode		1	2	3	4	5
Period (seconds)		0.880	0.288	0.164	0.106	0.073
Ratio of Period to 1st Mode Period		1.000	0.327	0.186	0.121	0.083
Participation Factor at Roof		1.31	-0.47	0.24	-0.11	0.05
Base Shear Participation		0.828	0.120	0.038	0.010	0.000
Mode Shape at Story Levels (normalized)	Roof	1.000	1.000	1.000	1.000	1.000
	7	0.938	0.550	-0.059	-0.857	-1.749
	6	0.839	-0.056	-0.942	-1.080	0.194
	5	0.703	-0.631	-0.921	0.526	1.674
	4	0.535	-0.961	-0.034	1.259	-1.068
	3	0.351	-0.933	0.883	-0.088	-1.139
	2	0.188	-0.625	0.990	-1.150	1.310
	1	0	0	0	0	0

Table 5–3. Seven-story building—transverse direction—summary of modal analysis.

Story		Forces (kips)				Shears (kips)				OTM (k-ft)			
Level	Wt kips	1	2	3	SRSS	1	2	3	SRSS	1	2	3	SRSS
Roof	1410	508	-330	170	629	508	-330	170	629	0	0	0	0
7	1460	494	-188	-10	529	1002	-518	160	1139	4420	-2871	1479	5474
6	1460	443	19	-166	473	1445	-499	-6	1529	13137	-7378	2871	15338
5	1460	371	216	-163	459	1816	-283	-169	1846	25709	-11719	2819	28394
4	1460	282	329	-6	433	2098	46	-175	2106	41508	-14181	1349	43884
3	1460	185	319	156	400	2283	365	-19	2312	59761	-13781	-174	61330
2	1830	125	267	219	367	2408	632	200	2498	79623	-10605	-339	80327
Ground		0	0	0	0					112131	-2073	2361	112175

Story	Acceleration (g)				Displacement (ft)				Interstory Drift (ft)				
	1	2	3	SRSS	1	2	3	SRSS	1	2	3	SRSS	Δa/hs
Roof	.360	-.234	.121	.446	.228	-.016	.003	.229	.014	.007	.003	.016	.0018
7	.338	-.129	-.007	.362	.214	-.009	.000	.214	.022	.010	.003	.024	.0028
6	.303	.013	-.114	.324	.192	.001	-.003	.192	.031	.009	.000	.032	.0037
5	.254	.148	-.112	.315	.161	.010	-.003	.161	.039	.005	.003	.039	.0045
4	.193	.225	-.004	.297	.122	.015	.000	.123	.042	.000	.002	.042	.0048
3	.127	.219	.107	.275	.080	.015	.002	.081	.037	.005	.001	.037	.0043
2	.068	.146	.120	.201	.043	.010	.003	.044	.043	.010	.003	.044	.0033
Ground	0	0	0	0	0	0	0	0					

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If the mode shapes are reasonably accurate, the calculated value of T_m will be the same at each story.

(i) If the calculated periods of vibration are substantially different than the values assumed in paragraphs (*a*) and (*b*), above, repeat paragraph (*e*) and adjust the modal forces and displacements in proportion to the new values of S_a.

(j) Compare the responses of the higher modes of vibration to the actions of the fundamental modes (e.g., refer to fig 2–10 and design example E–1). This includes story shears, story accelerations (i.e., story force divided by story weight), story overturning moments, and interstory displacements. If all the higher mode responses are small relative to the fundamental mode, they can generally be omitted from the analysis. If in no case the square-root-of-the-sum-of-the-squares (SRSS) of all the modes is less than 10 percent greater than the fundamental mode, it can be assumed that the higher modes are negligible in the overall design.

(4) *High-rise buildings.* As buildings get taller, the higher modes of vibration become more significant, relative to the fundamental modes (refer to para 2–5*c* and figures 2–9 and 2–10 for examples). These buildings generally require the use of computer programs that can calculate the dynamic characteristics (e.g., periods, mode shapes, and participation factors), as well as the member stresses and story displacements.

(5) *Irregular buildings.* Buildings that have vertical discontinuities, that are irregular in plan, that have large horizontal eccentricities (center of mass not coincident with center of rigidity), or have other irregularities will generally require the aid of computer programs to determine the dynamic characteristics, member stresses, and story displacements. When horizontal eccentricities exist, the analysis must be in three dimensions to account for the twisting deformations and the lateral deformations normal to the direction of the seismic forces. Refer to paragraph 5–4*d*, below, for use of three-dimensional computer programs.

b. Mathematical modeling of structural components. The results of a lateral-force analysis can be very sensitive to the assumptions made for the stiffness of the structural elements when constructing a mathematical model of the structure. As the stiffness is overestimated, the period of vibration shortens and the displacements reduce. However, a shorter period may possibly attract higher forces. When the stiffness is underestimated, periods lengthen, lateral displacements increase, and lateral forces may be reduced. When the relative rigidities of various lateral-force-resisting elements are not accurately utilized, there can be a great amount of uncertainty in the torsional characteristics of the structure. The effects of nonstructural elements, as well as structural elements not part of the lateral-force-resistant system, can have a significant effect on the response of the overall structure to earthquake ground motion. Therefore, it is important to account for possible inaccuracies in the mathematical model. When there are uncertainties, an attempt should be made to envelope the possibilities to assure good performance of the structure in case of an earthquake. The stiffness characteristics may vary with amplitude of lateral motion, thus the model used for a code design level analysis may vary from the model that represents the yield level capacity or the ultimate post-yield capacity. For an elastic analysis, the following factors should be considered:

(1) Gross concrete section properties are considered appropriate for modeling the stiffness of reinforced concrete members.

(2) The effects of column widths and beam depths on the rigidity of frames should be evaluated. This is particularly important for concrete frames or for steel frames with relatively deep members and short spans or low story heights.

(3) The effects of the floor slab system acting compositely with the frame beams or girders. Although the composite action may have an insignificant effect in resisting negative moments, it provides a significant contribution to the effective beam moment of inertia for positive moments and increases the stiffness of the beams acting as members of a rigid frame. In most cases, the beams will be modeled as prismatic members and engineering judgement will be required to determine an effective portion of the floor system to be modeled compositely with the beams. This composite action is used in the model of calculate the dynamic characteristics, but should be reevaluated for member design to resist negative moments.

(4) The effects of structural elements that are not included in the lateral-force-resisting system. This may include flat-slab and column systems and structural steel frames with standard connections. The effects of these elements on the stiffness of a building with shear walls or braced frames may properly be ignored, but they may have a significant effect on the stiffness of a building with a moment frame lateral-force-resisting system. In the latter case, the moment frames will be designed to resist 100% of the lateral forces, but the modeled stiffness of the frames will be adjusted to reflect the additional stiffness of the above elements, including any torsional effects due to asymmetry in the location of elements.

(5) The effects of relatively rigid nonstructural elements, such as masonary partitions, will be evaluated. If the stiffness of these elements is significant as compared to the stiffness of the assumed lateral-force-resisting system, the elements will be designed and reinforced as shear walls or will be isolated from the structural system by means of expansion joints at the sides and top of the element.

(6) Evaluate the effects of assumptions for modeling shear walls of various cross-sections. For example, the relative stiffnesses of an L-shaped wall and a wall that consists of a single plane. Also, the relative stiffness of a shear wall system and a moment frame system.

c. Two-dimensional computer programs. The designer must be familiar with all of the features and limitations of computer programs used for the design and analysis of buildings. A two-dimensional computer program essentially places all the lateral-force-resisting structural frames and shear walls within a single vertical plane and analyzes for lateral motion within that vertical plane. In a sense, each of the lateral-force-resisting column lines of the building are linked end-to-end. The two-dimensional analysis does not allow for any rotation about a vertical axis of the building (i.e., ignores horizontal torsion) and does not allow lateral sidesway normal to the direction of the applied force. The two-dimensional computer programs are applicable to buildings that are generally symmetrical in plan and are not subject to torsional deformation.

(1) *Features and limitations.* There are a variety of two-dimensional computer programs, each having certain features and limitations, such as the following:

(a) Dynamic characteristics. Some computer programs will calculate member forces and lateral deformations, but do not calculate the

periods or mode shapes of the structure. These programs can be used for low- to moderate-rise buildings where only the fundamental mode of vibration is required. The fundamental period and mode shape can be calculated and the effects of higher modes can be approximated by procedures outlined in paragraph 5-4*a*. However, computer programs are available that will calculate periods and mode shapes for all the modes of vibration.

(b) Axial, shear, and flexural deformations. Some computer programs are limited on the degrees of element deformations. Beams are generally considered as flexural elements. Some computer programs also account for shear deformation. Shear and flexural deformations are generally accounted for in column elements, but not all programs account for axial deformation. Axial column deformation can be significant in high-rise buildings however, caution must be used when applied to gravity loads because of the sequence of construction. Shear walls are generally analyzed for shear and flexural deformations.

(2) *Number of modes and use of participation factors.* In general, the first three modes of vibration in each horizontal direction of a building are sufficient for the model analysis. For tall buildings or for buildings with vertical irregularities, a greater number of modes may have to be analyzed. A review of the participation factors for the first three modes will give a good indication if more are required. The sum of the participation factors (PF_{xm}) for all the modes at a particular story (x), as calculated from equation 4-1, equals unity. Also, the sum of all the modal base shear participation factors, (α), as defined in equation 4-2 will equal unity. Therefore, if the sum of the participation factors for the first three modes is within 10 percent of unity, it can generally be assumed that all the major modes have been included. For an example, refer to table 5-2. The sum of the participation factors at the roof for three modes equals 1.08 (i.e., 1.31 − 0.47 + 0.24) and the sum of the base shear participation factors is equal to 0.986 (i.e., 0.828 + 0.120 + 0.038). Both 1.08 and 0.986 are within 10 percent of the value of 1.0.

(3) *Check static equilibrium.* Some computer programs present the results for each individual mode and others only present the results in modal combinations. Once the modes have been combined, it is not possible to check the statics for the overall building or for localized areas, such as at a beam-column joint. Therefore, static checks must be made prior to making the modal combination. Spotchecks at a variety of locations should always be made to assure that static equilibrium is maintained. These checks are made not only to confirm the validity of the computer program, but they also alert the designer to possible irregularities or to the possibility of data input errors.

(4) *Accidential torsion.* The two-dimensional computer analyses do not account for torsional motion due to horizontal eccentricities. However, the effects of horizontal eccentricities or the requirement for accidential torsion can be approximated by hand calculations in conjunction with the results obtained from the computer analysis. The horizontal torsional moment can be calculated from the product of the story shear and the assumed eccentricity. The torsional moment can then be distributed to the lateral-force-resisting elements in proportion to the product of their relative rigidities and distances from the center of rotation (Kd) divided by the torsional moment of inertia (ΣKd^2). The forces obtained from the computer can then be proportioned upward to account for the additional forces due to torsion. The minimum torsional eccentricity that is to be applied to a structure is equal to 5 percent of the maximum building dimension (Basic Design Manual, para 3-3(E)4). A rational alternative to this requirement is to calculate accidental torsions by using eccentricities that result by moving the center of mass of each story 5 percent of the maximum building dimension to either side of its calculated position (Basic Design Manual, para 5-2*d*(4)). An example is included in design example E-2.

(5) *Flexible horizontal diaphragms.* Two-dimensional computer programs assume that the diaphragms are infinitely rigid. In some buildings, the horizontal diaphragms may exhibit some flexibility relative to the vertical lateral-force-resisting elements. For very flexible diaphragms, the forces should be distributed to the vertical lateral-force-resisting elements by means of tributary areas. When a limited amount of flexibility is anticipated, the forces on the less rigid elements of the rigid diaphragm model should be increased to account for possible additional forces due to tributary area distribution. Some judgment decisions are required. When there is difficulty in determining the proper distribution of forces, a three-dimensional analysis that accounts for diaphragm flexibility may be required.

d. Three-dimensional computer programs. Three-dimensional computer programs become much more complex than the two-dimensional programs, and more care must be taken to fully

understand their features and limitations. Three-dimensional programs can account for rotation about a vertical axis and horizontal movement in any direction. Some programs, usually those using finite element procedures, can allow for flexibility in the horizontal diaphragm. The contents of paragraph *c*, above, in general also apply to three-dimensional programs. Additional comments, which apply to three-dimensional programs, follow:

(1) *Features and limitations.* There are a variety of three-dimensional computer programs, each having certain features and limitations, such as the following:

(a) Three-dimensional compatibililty. Some three-dimensional computer programs were developed as extensions of two-dimensional programs. The three-dimensional features are determined by combining the components of two-dimensional analyses. In some cases, where a structural element is part of both a transverse and longitudinal lateral-force-resisting system, compatibility of common actions from both directions of force is not maintained (e.g., axial forces and vertical deformations in a column common to two intersecting systems are not truly compatible).

(b) Horizontal eccentricities. Additional care must be taken in preparing the data for three-dimensional computer programs. Torsional characteristics of a building are sensitive to the size and location of the story weights and the rigidity properties of lateral-force-resisting elements on the horizontal story plane. In some computer programs, mass moments of inertia are required. In other programs, the masses are distributed on the horizontal planes. Assumptions used in modeling a variety of shear walls and frames can be critical in the evaluation of torsional properties and horizontal eccentricities; therefore, methods to envelope the uncertainties as discussed in paragraph 5-4*b* should be investigated.

(c) Modal combinations. Because the computer programs allow for three-degrees-of-freedom (longitudinal, transverse, and rotational), combining the modes in three-dimensional analysis becomes substantially more complex than combining modes for two-dimensional analysis. In some cases, the use of the SRSS can give erroneous results, especially when the loads are applied in a direction not parallel to the major axes. Therefore, other procedures for combining the modes are required. The designer must be aware of the procedures and pitfalls that may be inherent in the computer program being used in relation to the building being analyzed.

(2) *Modes and participation factors.*

(a) Mode identification. In three-dimensional analyses, it is sometimes difficult to identify the characteristics of the various modes of vibration. For a regular building, the first three modes will generally include the fundamental modes that represent primary motion in the translational transverse direction of the building, the translational longitudinal direction of the building, and the rotational torsional action of the building. The first nine modes listed in the order of decreasing lengths of period will generally include the first three modes of each of those directional motions. However, for unusual buildings, the sequence of the modes may be highly irregular. For example, a building with very low torsional rigidity will have torsional modes with long periods of vibration, thus the translational modes may not be identified until after several torsional modes are calculated. Another example is in buildings with flexible diaphragms. If the diaphragms are more flexible than the overall structure, the modes for each of the flexible diaphragms will be calculated before the primary building modes are identified. Each of these examples would indicate that the building may have some undesirable characteristics or that there may be an error in the modeling of the building. Modes can be identified by plotting the mode shapes in three-dimensional representations.

(b) Participation factors. The concept of participation factors also becomes more difficult to interpret in three-dimensional analyses; therefore, the guidelines given in paragraph 5-4*c*(2) to identify the number of modes required for analysis may not be applicable for buildings with unusual three-dimensional characteristics. For each direction of applied earthquake forces there will be a major component in the direction of motion, a translational component normal to the direction of applied forces, and a rotational component. The participation factors, based on the mode shapes (ϕ) in the direction of applied motion will not add up to 1.0, as occurs in the two-dimensional programs, because of the contribution of the other components of motion. If the base shear participation factors (α) do not add up to within 90 percent of unity, then all of the values of the modal analysis will be increased proportionately to satisfy the 90-percent requirement.

e. Stresses and load combinations. The loads on the structural elements resulting from the

modal analysis procedure for EQ-I must be combined with the gravity loading to determine if the structure has remained essentially elastic.

(1) *Load combinations.* The seismic loads due to the actions of EQ-I will be combined with gravity loads in accordance with equations 4-6 and 4-7. Equation 4-6 is used when the gravity loads are in the same sense as the seismic loads (e.g., both sets of loads result in compression in a column or negative bending moments in a beam). Equation 4-7 is generally used when there is a potential for load reversal (e.g., tension in column due to seismic loading may be greater than compression due to minimum dead load, or the positive bending moment due to seismic loading is greater than the negative bending moment due to minimum dead load). The 1.2 and 0.8 coefficients for the dead load are established to represent possible vertical seismic accelerations as well as some uncertainties in the actual dead weight of the structure. Equation 4-8 is a special case for use on horizontal prestressed components that are especially sensitive to upward vertical accelerations.

(2) *Elastic capacity ratio.* The elastic capacities of the structural elements are computed in accordance with the provisions of paragraph 4-3*f*. The elastic capacities of the structural elements will generally be equal to or greater than the load combinations determined in paragraph (1) above. Some exceptions are permitted in accordance with paragraph 4-3e(1). The *elastic capacity ratio* is a term used to determine if there is any reserve elastic capacity remaining beyond the demands of EQ-I. It is calculated from equations 4-6 and 4-7 as follows:

$$\text{Elastic capacity ratio} = (EC - 1.2D - 1.0L) \div 1.0E \qquad \text{(eq 5-4)}$$

or

$$= (EC + 0.8D) \div 1.0E \qquad \text{(eq 5-5)}$$

whichever is less. Note that the elastic capacity is reduced by gravity loads when they are in the same sense as seismic loads per equation 5-4 and the elastic capacity is increased by minimum dead loads when they are in the opposite sense of seismic loads per equation 5-5. The *elastic capacity ratio* of the overall structure is equal to the lowest value for any group of major structural elements. It is used to define when first major yielding occurs and to establish peak floor accelerations and response spectrum for nonstructural elements in paragraph 6-4. For its use in the EQ-I analysis, refer to paragraph 5-5*b*.

f. Displacements and drifts. Lateral story displacements for each mode of vibration are calculated from equation 4-5, and examples are given in appendix E, design example E-1. Maximum story displacement by the SRSS method. CAUTION: the maximum interstory drifts cannot be obtained from the maximum story displacements. The interstory drifts must first be obtained for each individual mode. The interstory drifts for each mode may then be combined by the SRSS method to obtain the maximum interstory drifts. It is these maximum interstory drifts that will satisfy the limitations of paragraph 4-3*e*(7)(*a*). The maximum story displacements are required for the criteria for building separations in paragraph 4-3*e*(7)(*b*). In three-dimensional analyses, should there be an appreciable amount of rotation of the horizontal diaphragms, the displacements and the interstory drifts at the outer limits of each floor level will be determined. If the portion of displacements due to rotation begins to approach the portion of the displacements due to translation (e.g., if the displacement at the outer edge of the building is greater than 1.5 times the displacement at or about the center of rotation), an evaluation of the potential for torsional instability will be investigated as outlined in paragraph *i* below.

g. Accelerations. Story accelerations for each mode of vibration are calculated from equation 4-3. The story acceleration is equal to the story lateral force (F_{xm} divided by the story weight (w_x). Maximum story accelerations may be obtained by the SRSS method. Floor accelerations are used to establish criteria for the design of elements attached to the floors of the building, as prescribed in chapter 6. In three-dimensional analyses, should there be an appreciable amount of rotation of the horizontal diaphragms, the accelerations at points of interest at various locations on each floor level will be determined. Modal accelerations at these locations can be calculated from the modal displacements determined in paragraph f above by equation 5-6, which is derived from equations 4-3 and 4-5:

$$a_{xm} = F_{xm}/w_x = \delta_{xm}\,(2\pi/T)^2 \div g \qquad \text{(eq 5-6)}$$

h. Overturning. The structure, that portion above the foundation interfacing with the supporting soil medium, will be designed to resist the overturning effects of the seismic loading. In some portions of the structure, the resulting forces may cause uplift at the foundation interface, thus creating an apparent overturning instability condition. However, structures designed for force levels substantially less than

those experienced during actual earthquakes have not exhibited this behavior. Although the state-of-the-art of earthquake engineering has not been able to establish a consistent recommendation for evaluating this condition, it is generally acceptable that buildings can be subjected to rocking on their bases, that the resulting displacements do not approach an incipient overturning condition, and that the maximum displacement is limited by the short time interval between load reversals. When the design engineer determines that uplift conditions exist, two basic choices exist: (1) tie down the foundation to prevent uplift; or (2) do not provide any additional restraint on the potential uplift. The decision requires some judgment of the engineer. If the foundation is tied down, the resulting forces on the structure will generally be increased in the event of a large earthquake because of the added rigidity of the overall structural system. If uplift is allowed to occur, the resulting seismic forces may actually be reduced because of increased energy absorption and the nonlinearity of the base rocking; however, the redistribution of loads to other portions of the foundation may cause some distress in the structure or at the foundation. When uplift is allowed to occur, the designer will provide justification for the assumed redistribution of loads and for the adequacy of the structure and foundation.

i. Horizontal torsional moments. Torsional-resisting elements, as part of the lateral-force-resisting system, should preferably be located at or near the periphery of the building to maximize torsional rigidity. When this cannot be accomplished or when there are large horizontal eccentricities, the structure must be analyzed for potential torsional instability.

(1) Compare the forces due to translational motion to the forces due to torsional motion for all lateral-force-resisting components. If the torsional portion is a substantial amount of the total design force (e.g., one-third of the total), then torsional stability will be evaluated.

(2) Review the mathematical modeling assumptions and calculations to evaluate the validity of the modeling techniques. Determine if uncertainties in assumptions would increase or decrease the torsional characteristics.

(3) Investigate the consequences of the worst-case conditions.

(4) Evaluate the feasibility of revising the lateral-force-resisting system to minimize the effects of horizontal torsional moments.

j. Minimum lateral forces requirements. Paragraph 4-3*d* requires a comparative study of the EQ-I forces and the static force criteria of the Basic Design Manual. If the EQ-I forces should be less than the adjusted Basic Design Manual forces, justification is required. This requirement is made to reduce the risk of error or misinterpretation of the seismic design procedures of this manual and applies to all building classifications. In lieu of a justification statement, the EQ-I forces may be increased by a value that results in net story shears at least 50 percent greater than the story shears determined from the minimum earthquake forces prescribed in the Basic Design Manual. The procedure is outlined in paragraph 5-3*d*(2)(*a*) through (*c*). The absolute lower limits of 3 percent and 2 percent apply to buildings with very long periods (e.g., T greater than 3 seconds).

5-5. Designing for EQ-II.

The structure will be analyzed to determine its ability to resist the forces and deformations caused by design earthquake EQ-II. At this point in the design, the initial design has been developed as outlined in paragraph 5-3*a*, and the structure modified, if necessary, to be able to withstand the forces of EQ-I elastically, as outlined in paragraph 5-4. Two procedures are presented for post-yield analysis provisions in paragraph 4-4 as acceptable methods for evaluating the capacity of the structure to resist the actions of EQ-II.

a. Method I: Elastic analysis procedure. This is an elastic analysis procedure that is essentially the same as the procedure outlined in paragraph 5-4 for EQ-I. The exceptions are noted.

(1) *Modal analysis procedure.* The procedure is the same as outlined in paragraph 5-4*a*. The spectral accelerations will generally be larger; however, there will be a higher percentage of damping and the periods of vibration may be slightly longer.

(2) *Mathematical modeling of structural components.* The comments of paragraph 5-4*b* generally apply; however, some modification to the modeling assumptions may be made.

(a) Allowances may be made to account for the reduced section properties of cracked or partially cracked concrete.

(b) Allowances may be made for flexibility at beam-column joints.

(c) Unless the floor slab system is integrated into the design of the beams and girders, composite action need not be considered.

(d) The effects of nonseismic frames should be reevaluated in regards to the larger deformations resulting from EQ-II. These ef-

fects would usually be ignored in the mathematical model unless they provide redundancy for the overall lateral-force-resisting system.

(e) The effects of nonstructural elements is not included in the mathematical model to calculate periods, displacements, and member forces. However, the possible detrimental effects of rigid nonstructural elements must be considered in the overall evaluation of the structure.

(f) The modification of modeling assumptions can result in the lengthening of periods of vibration by 25 percent to 50 percent.

(3) *Stresses and load combinations.* The loads on the structural elements resulting from the modal analysis procedure for EQ-II must also be combined with the gravity loading. However, the load factors on dead and live loads have been revised in accordance with equations 4-9 and 4-10. Only the actual dead load need be considered, and the design live load may be reduced to a value that is consistent with actual live loads that are likely to be in place at the time of a severe earthquake. This reduced gravity loading is justified on the basis of the probability that it is unlikely that both maximum live loads and maximum earthquakes will occur at the same time.

(4) *Inelastic demand ratios.* The Method 1 evaluation procedure is based on the assumption that EQ-II will result in a number of lateral-force-resisting elements being stressed beyond their elastic limit yield capacities.

(a) The calculated forces on the structural elements are obtained from an elastic analysis. Therefore, these are the force *demands* of EQ-II if the structure had remained elastic.

(b) The *capacities* are defined as the strength of the element at the point of yielding.

(c) The ratio of the *demand* to the *capacity* (i.e., the *inelastic demand ratio*) is an indication of the ductility that may be required for the structural element to withstand the forces of EQ-II. As the first elements of the overall structure begin to yield (i.e., inelastic demand ratio exceeds 1.0), forces will be redistributed to other elements of the lateral-force-resisting system. The limiting values of *inelastic demand ratios* for structural elements prescribed in table 4-2 have been established as acceptable limits for a structural system that has a reasonable amount of redundancy and is not subjected to premature vertical or torsional instability or to a premature mechanism at a single story level. Possible weak links in the overall structural system are detected by investigating the distribution of the *inelastic demand ratios* that exceed a value of 1.0. Conditions to be evaluated are listed in paragraphs 4-4c(5) *(b)*, *(c)*, and *(e)* and are discussed in paragraphs *(d)*, *(e)*, and *(f)*, below.

(d) Unsymmetrical yielding on a horizontal plane. This provision is used to check for the possibility of torsional instability, as discussed in paragraph 5-4*i*. For example, if all the *inelastic demand ratios* on the north side of the structure were greater than 1.0, and all the ratios on the south side were less than 1.0, a potential for torsional instability exists. Yielding of the north side will reduce the stiffness of that side of the building relative to the south side, thus the center of rigidity moves to the south. If this condition increases the horizontal eccentricity of the building, torsional moments increase geometrically and the potential for collapse is present.

(e) Hinging of columns at a single story. This provision is used to check for the possibility of an unstable soft story. For example, if *inelastic demand ratios* were equal for about 1.5 at the tops and bottoms of 80 percent of the columns for the first story of a multistory building and *inelastic demand ratios* for columns at every other story were less than 1.0, the potential for instability at the first story exists. Because the columns are yielding only at the first story, all the inelastic energy will have to be absorbed at that level. This subjects the first story to the possibility of excessive interstory displacements.

(f) Unusual distributions of inelastic demand ratios. This is a more general case of paragraphs *(d)* and *(e)*, above. This provision is used to check the efficiency of the overall lateral-force-resisting system. If a limited number of structural elements have large *inelastic demand ratios* and the remainder of the elements have ratios less than 1.0, it might be prudent to consider some structural modifications to reduce the potentially high demands on a small number of structural elements.

b. Method 2: Capacity spectrum method. This is an approximate inelastic analysis procedure. The ability of the structure to resist the forces and deformations caused by EQ-II is determined by a graphical method. The procedure requires the construction of two curves. One curve represents the *capacity* of the structure to resist lateral forces and the other curve represents the *demand* of the ground shaking. The *capacity* curve is developed from a force (F or V) versus displacement (δ) relationship of the overall structure. Modal analyses are used to determine

levels of excitation to yield structural elements. The *capacity* is defined by the forces and displacements of the fundamental mode. The force-displacement curve can be converted into a spectral acceleration (S_a) versus period (T) curve (i.e., a capacity spectrum) by means of equations 4–3, 4–4, and 4–5. The *demand* of the ground shaking is represented by an EQ–II response spectrum curve. This curve is a composite of the two damping values (elastic-linear and post-yield) determined from table 4–1. The *capacity* curve and the *demand* curve are plotted on the same graph; their intersection is considered to be the reconciliation between demand and capacity. A sample building, six stories and a 66-foot height, is used for illustration. Figure 5–5 shows the force-displacement capacity curve for the sample building. It plots the base shears (V) and roof displacements (δ_n). In table 5–4, the V and δ_n values are converted to spectral accelerations (S_a) and periods (T) using equations 4–4 and 4–5 with the participation factors (PF_n and α) for the fundamental mode of vibration. The capacity curve is plotted on figure 5–6 with two response spectra representing EQ–II. The 5-percent damped demand curve is used for the elastic capacity (T < 0.80 sec) and the 10-percent damped demand curve is used for the ultimate capacity (T < 1.4 sec). A transition curve is drawn between T = 0.80 sec and T = 1.4 sec. Following are guidelines for constructing the capacity curve using a step-by-step method and approximating the lateral displacements and drifts.

(1) General procedures for constructing the capacity curve. The capacity curve is a simplified global representation of the building capacity. As localized yielding occurs (e.g., bending at the end of a girder), the overall (or global) characteristics of the building are modified. If the localized yielding is at a critical structural element, the global characteristics may change significantly. Conversely, if the localized yielding is at a redundant location, the change to the global characteristics may be insignificant. For single-story buildings and low-rise buildings up to about 5 stories, the modal analysis procedure for constructing the capacity curve can generally be limited to the fundamental mode of vibration.

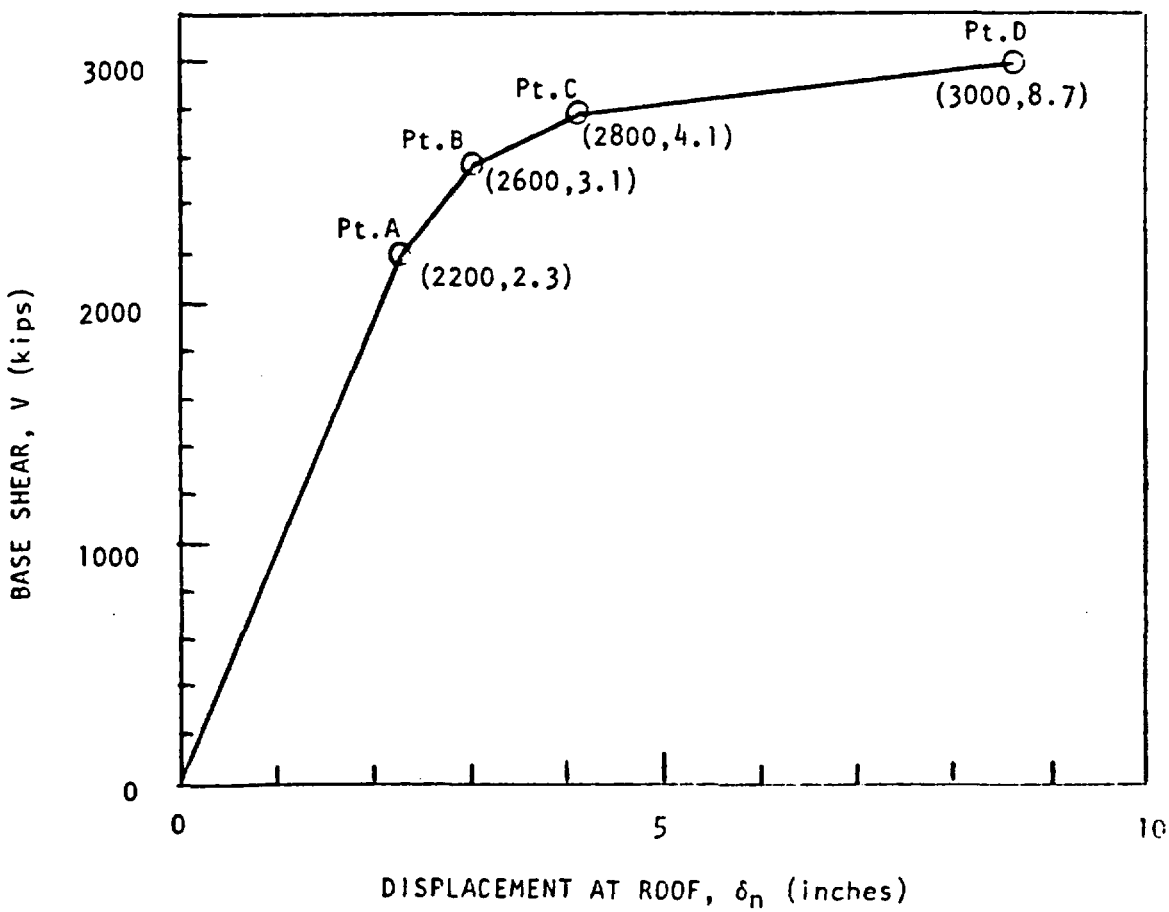

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Figure 5–5. Force-displacement capacity curve.

Table 5-4. Conversion of V and δ_n to S_a and T.

Point	V (kips)	δ_N (in)	$V/_W$	PF_N	α	S_a (g)	S_d (in)	T (sec)
A	2200	2.3	0.22	1.30	0.78	0.280	1.77	0.80
B	2600	3.1	0.26	1.28	0.80	0.325	2.42	0.87
C	2800	4.1	0.28	1.28	0.80	0.350	3.20	0.97
D	3000	8.7	0.30	1.26	0.83	0.361	6.90	1.40

V/W: V = Base Shear, W = Weight = 10,000 Kips

δ_N = Lateral roof displacement due to V

$PF_N = (\Sigma m\phi)\ (\phi_N)/(\Sigma m\phi^2)$, modal roof participation factor (eq. 4-1)

$\alpha = (\Sigma m\phi)^2/(\Sigma m)(\Sigma m\phi^2)$, effective modal weight (eq. 4-2)

S_a = Spectral acceleration = $V/_W \div \alpha$ (eq. 4-4)

S_d = Spectral displacement = $\delta_N \div PF_N$ (eq. 4-5)

$T = 2\pi\sqrt{S_d/(S_a)(g)}$, fundamental period of vibration (eq. 4-5)

$\Sigma m\phi$ = Summation of story mass times mode shape factor from the roof to the base of the building

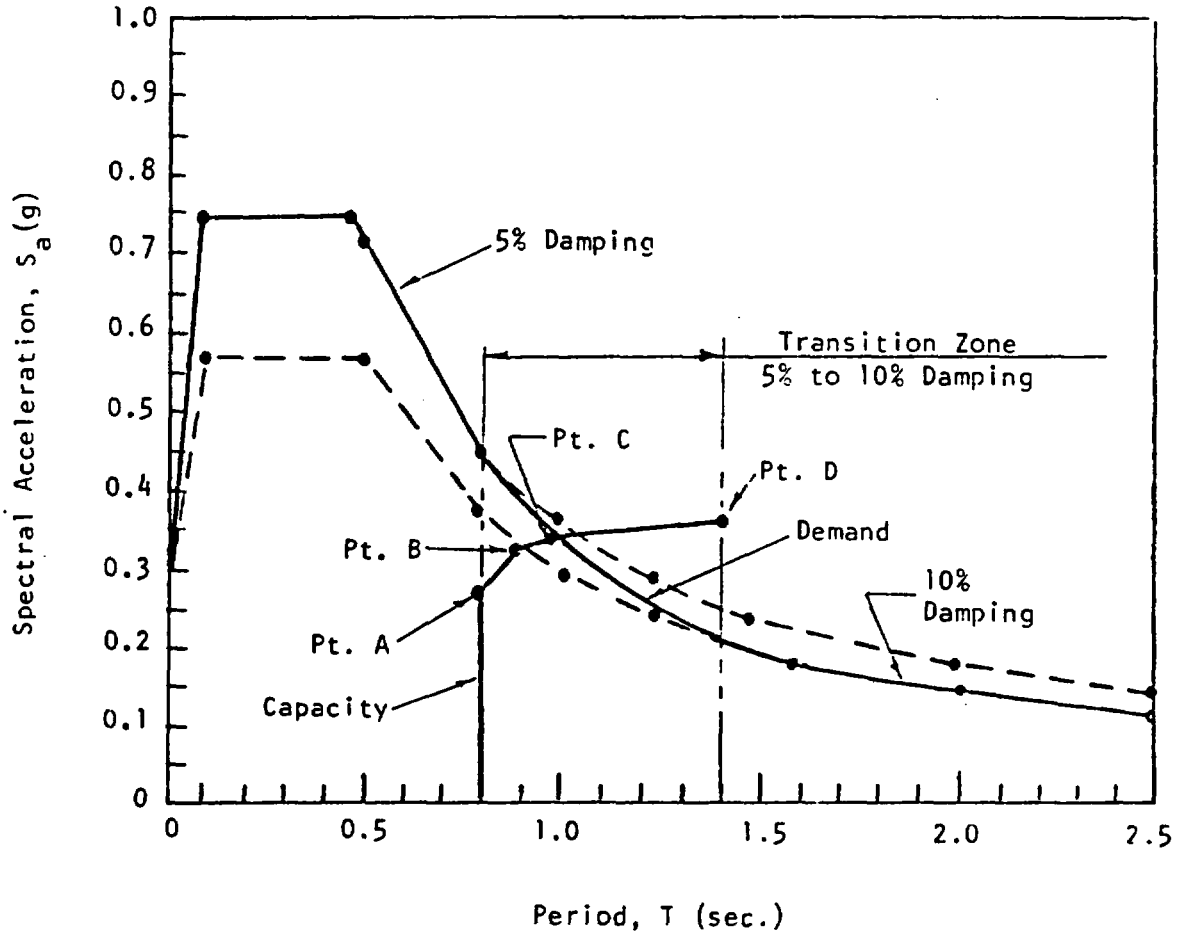

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Figure 5-6. Capacity spectrum method.

For taller buildings, effects of higher modes of vibration may become significant, thus a multimode analysis may be required. The results from the EQ-I design can be used to determine the effects of the higher modes and the necessity of using them in the EQ-II analysis., The capacity curve is developed by a step-by-step procedure, using superposition, where the structure is laterally distorted to a limiting value, frozen in that position, local yielding elements are relaxed, and the structure is laterally distorted to a new value. The procedure is repeated until an ultimate limit is reached. The capacity curve is constructed by means of superposition of straight lines. The period and stiffness characteristics are determined from the secant modulus drawn from the origin to the various points on the force-displacement curve.

(2) Single-mode capacity curve. If it is determined that only the fundamental mode is required (i.e., higher modes are insignificant), the shape of the ground motion response spectrum is not required for the construction of the capacity curve. The following procedure can be setup in tabular form:

(a) Determine the *elastic capacity* (EC) for each structural element (e.g., negative and positive moment capacities at each end of each girder, interation diagrams at ϕ = 1.0 for each column, and shear and moment capacities of shear walls at various key locations). These capacities are defined as the strength of the element at the point of yielding and should be available from the EQ-I DESIGN.

(b) Determine the *net capacity available for earthquake loading* in each element using the EQ-II load combination criteria of equations 4-9 and 4-10, paragraph 4-4e(1). For example, equation 5-7 for negative moments and equation 5-8 for positive moments at ends of girders. Note that the net earthquake capacity is reduced by gravity loads when they are in the same sense as seismic loads per equation 5-7 and the net earthquake capacity is increased by dead loads when in the opposite sense per equation 5-8.

Net earthquake capacity $= EC - D - 0.25L$ (eq 5-7)

Net earthquake capacity $= EC + D$ (eq 5-8)

(c) Divide the net earthquake capacities for each element by the corresponding earthquake loads determined in the EQ-I design. This gives local elastic capacity ratios for each element. The lowest ratio, or group of ratios with a 10-percent variation, establishes the *global elastic capacity ratio* for the structure as described in paragraph 5-4*e*(2), adjusted for EQ-II load factors.

(d) Establish the point of *initial major yielding*, the first point on the capacity curve, by multiplying the EQ-I design base shear and lateral roof displacement by the *global elastic capacity ratio* for the structure. This point is represented as point A by V = 2200 kips and δ_n = 2.3 inches for the sample six-story building characterized in table 5-4 and figure 5-5.

(e) Determine the first *post-yield segment* of the capacity curve. The structure is essentially frozen at the point of initial major yielding. The balance of net capacity in each element still available for additional earthquake loading is tabulated. Elements that are at or near (e.g., within 10 percent) their yield capacities are modeled as plastic hinges (e.g., beam elements might have their moments of inertia reduced to 5 percent of their elastic values). Lateral forces proportional to the fundamental mode shape are applied to the revised mathematical model. For the sample six-story building, the base shear of the applied forces was 1000 kips. The resulting forces on the elements were compared to the balance of net earthquake capacities and lateral displacements were calculated. It was determined that 40 percent of the applied loads will form a new group of yielding elements. A second point on the capacity curve was determined at V = 2600 kips and δ_n = 3.1 inches (2200 kips at point A plus 40 percent of 1000 kips and 2.3 inches at point A plus 40 percent of 2.0 inches), represented by point B in table 5-4 and figure 5-5.

(f) Determine sequential post-yield segments on the capacity curve by repeating the procedure in (*e*) above (e.g., points C and D in table 5-4 and figure 5-5 using revised mode shapes and mathematical models).

(g) The procedure is repeated until a failure mechanism, instability, or excessive deformations occur. Rotational ductility demands can be approximated by using M/EI diagrams of the yielding girders, taking into account the reduced EI's used in the yielding mathematical model. Ductility demands for flexure should not exceed 2 times the Inelastic Demand Ratios of table 4-2, and for all other conditions they should not exceed the values shown in table 4-2. Interstory displacements are determined by superposition of the lateral story displacements of the sequential models. For the sample six-story building, the ultimate global capacity of the structure is represented by point D at V = 3000 kips and δ_n = 8.7 inches in table 5-4 and figure 5-5.

(h) Deterine lateral displacements and drift demands. The capacity curve is converted to S_a and T coordinates and superimposed on the EQ-II response spectrum curve. If the curves do not intersect, irreparable damage or collapse of the structure is anticipated for EQ-II. If the curves do cross, the intersection can be used to approximate the response of the structure to EQ-II. For the sample six-story building, data from table 5-4 are shown in figure 5-6. The intersection of the capacity and demand curves is about S_a = 0.35g and T = 10.0 seconds. The lateral story displacements at this intersection are calculated from equation 4-5.

$$\delta_n = PF_nS_a\,(T/2\pi)^2 g$$
$$= 1.28 \times 0.35\,(1/2\pi)^2\,386 = 4.38 \text{ inches}$$

The roof displacement equals about 4.4 inches for a six-story building, 66 feet high. Maximum interstory displacements can be obtained from a composite deflected shape estimated from the sequential incremental analysis done above, or by proportioning the interstory drifts by the ratio of the EQ-II displacements to the EQ-I displacements. For the sample building, the average interstory drift is 0.73 inches. The maximum interstory drift, which is at the second story, equals 1.1 inch or 0.0083 times the story height. Thus, it satisfies the requirements of drift (i.e., less than 0.010) as prescribed in paragraph 4-4*e*(2)*(a)*.

(i) The results of this procedure give an estimate of the inelastic response of a building to a severe earthquake. In general, it will result in lower force levels and larger displacements than the results of Method 1 in paragraph 5-5*a*. Neither procedure is necessarily more accurate than the other; however, an evaluation of both procedures should give the designer enough insight to determine the weak links of the structural system, evaluate the potential for instability, and suggest needs for possible structural modifications.

(3) *Multi-mode capacity curve.* If it is de-

termined that the higher modes are significant, a multi-mode analysis is required. The *shape* of the ground motion response spectrum is required for the construction of the capacity curve. In general, the shapes of the EQ-I and EQ-II response spectra are similar; therefore, the EQ-I response spectrum is usually used because of data available from the EQ-I analysis. The procedure for constructing the multi-mode capacity curve is the same as the procedure for the single-mode capacity curve, paragraph (2) above, with the following exceptions:

(a) Same as paragraph (2) *(a)*.

(b) Same as paragraph (2) *(b)*.

(c) Same as paragraph (2) *(c)*, except that the corresponding earthquake loads determined in the EQ-I design are determined by a multi-mode analysis.

(d) Same as paragraph (2)(d), except that only the *fundamental mode component* of the EQ-I design base shear and lateral roof displacement are multiplied by the global elastic capacity ratio. For example, assume the data in table 5-3 represents the *initial major yielding* for the seven-story building. The multi-mode base shear is 2498 kips, but the fundamental mode component is 2408 kips. The multi-mode roof displacement is 0.229 feet and the fundamental mode roof displacement is 0.228 feet. Although 2498 kips represents the forces used to determine the *initial major yielding* in the building, the values of 2408 kips and 0.228 feet represent the "point A" used in the capacity spectrum (i.e., such as table 5-4 and fig 5-5).

(e) Same as paragraph (2) *(e)*, except that the lateral forces are applied by means of a multi-mode response spectrum analysis (e.g., use EQ-I response spectrum). If the EQ-I response spectrum, with a peak ground acceleration of 0.10g is applied to the revised mathematical model and it is determined that 40 percent of the resulting multi-mode forces will form a new group of yielding elements, the second segment of the capacity curve is determined by using 40 percent of the *fundamental mode component* of base shear and lateral roof displacement. This is the same as finding the spectral acceleration for the first mode period on a response spectrum that has a peak ground acceleration of 0.04g (i.e., 40 percent of 0.10g). First-mode spectral acceleration and period can be converted to base shear and roof displacement by the formulas shown in table 5-4. As in paragraph *(d)* above, the forces in the elements are determined by the multi-mode analysis, but the capacity spectrum is represented by the fundamental mode component.

(f) Same as paragraph (2) *(f)* with the exceptions noted in *(e)* above.

(g) Same as paragraph (2) *(g)*, except that the interstory displacements determined by superposition of fundamental modes represented in the capacity curve must be increased proportionally to represent the multi-mode analysis. For example, the interstory drifts between the sixth and seventh stories in table 5-3 are 0.024 feet for the multi-mode analysis and 0.022 feet for the fundamental mode. Therefore, interstory displacements determined by superpositions of the sequential fundamental modes will be increased by a factor of 0.024/0.022 equals 1.09. Between the third and fourth floors, the values are the same and no correction is required.

(h) Same as paragraph (2) *(h)* except that the lateral displacements that represent the first mode component must be increased proportionally to also represent the multi-mode components. For example, in table 5-3 roof displacements will be increased by a factor of 0.229/0.228 = 1.004.

(i) Same as paragraph (2) *(i)*.

(4) Variations of the procedures outlined above for constructing a capacity curve are acceptable with justification.

c. Displacements and drifts. Lateral displacements and drift limits are prescribed in paragraph 4-4*e*(2). Methods of calculating the displacements are described in paragraphs 5-5*a* and 5-5*b*. In general, the results of Method 2 will give larger displacements than the results of Method 1; however, the reverse can occur in some cases. If the differences of the two methods will effect the outcome of the design of the structure, a reevaluation of the procedures or assumptions will be made to justify an acceptable solution. A secondary effect of lateral displacements, when combined with gravity loads, is the possibility of P-delta instability. Guidelines are given in paragraph 5-5*d*.

d. P-delta effects. The P-delta effects in a given story are due to the eccentricity of the gravity loads above the story. If the story drift due to the lateral forces are *delta*, the bending moments in the story would be augmented by an amount equal to *delta* times the *gravity load* above the story. The ratio of the P-delta moment to the lateral-force story moment can be designated as a stability coefficient, θ. If the stability coefficient is less than 0.10 for every story, then the P-delta effects can be considered insignificant. If, however, the stability coefficient, θ, exceeds 0.10 for any story, then the P-delta effects for the whole building must be determined by a rational analysis.

CHAPTER 6
NONSTRUCTURAL ELEMENTS

6-1. Introduction.

This chapter prescribes the criteria for nonstructural elements that must remain intact or functional after a major seismic disturbance. The provisions of this chapter include the determination of the seismic forces to be applied to the elements, the determination of the deformations that the elements will withstand, and the criteria for the design of architectural, mechanical, and electrical elements to resist the prescribed forces and deformations. The criteria and design standards of this chapter provide a dynamic analysis approach to the seismic design of nonstructural elements and their anchorages that may be used in lieu of, or as supplements to, the provisions of chapters 9 and 10 of the Basic Design Manual.

6-2. General requirements.

The elements and their anchorages will be designed to resist the forces and deformations caused by the motion of the building in which they are placed, as prescribed in paragraph 4-2e. The effects of the nonstructural elements on the performance of the structure must also be considered.

a. Under the conditions of EQ-I, the elements will be designed to resist the applied forces and deformations without exceeding yield stresses.

b. Under the conditions of EQ-II, the elements will be analyzed for their ability to withstand the applied forces and deformations, such that: (1) they will not collapse or endanger life safety when subjected to the provisions of paragraph 6-4; and (2) they will remain functional, if required, in accordance with the provisions of paragraph 6-7.

6-3. EQ-I provisions.

The elements in or on the structure will be designed to resist the forces and deformations caused by the response of the structure to EQ-I, in accordance with criteria presented in this paragraph.

a. Method of analysis. The total design force representing earthquake effects will be determined from the maximum floor (or roof) accelerations of the building and from a design floor (or roof) response spectrum based on 2 percent damping for the elements. This requirement does not prohibit the use of properly substantiated time history response analysis procedures.

b. Maximum floor acceleration. The maximum floor accelerations will be determined from the modal analysis methods prescribed in chapter 4. Modal story accelerations will be determined using the equation 6-1:

$$a_{xm} = PF_{xm}S_{am} \qquad \text{(eq 6-1)}$$

where:

a_{xm} = modal story acceleration at level x for mode m.

PF_{xm} = modal participation factor as determined by equation 4-1.

S_{am} = spectral acceleration for mode m.

Equation 6-1 is derived from equation 4-3, where $a_{xm} = F_{xm}/w_x$.

(1) For 2D analyses, the maximum floor acceleration will be determined from the SRSS combination.

$$(a_x)_{max} = \sqrt{\Sigma a_{xm}^2} \qquad \text{(eq 6-2)}$$

(2) For 3D analyses, the maximum floor acceleration will be determined by an approved method to account for a rational combination of the modal values. When torsional motion is significant, relative to translational motion, variations of modal accelerations within the plane of the floor level will be considered. Guideline procedures are included in paragraph 5-4.

c. Design floor response spectrum. A procedure for approximating a design floor response spectrum is outlined herein. This procedure uses the peak modal accelerations determined from equation 6-1, the modal periods of vibration of the structure in accordance with the provisions of chapter 4, and the magnification factor (M.F.) curve shown in figure 6-1 (reproduced from Basic Design Manual fig 10-2). For each floor of the structure, the following procedure is used:

(1) For each significant modal period of vibration (T_m) and from the dynamic responses of the structure, calculate the story accelerations, a_{xm}, using equation 6-1 for the story where the equipment is supported (see table 6-2 for an example at the roof level).

(2) Establish a coordinate system with S_{fa} (floor spectral acceleration), the ordinate, and T_a (period of the equipment or architectural appendage), the abscissa. Develop the floor response spectrum as follows:

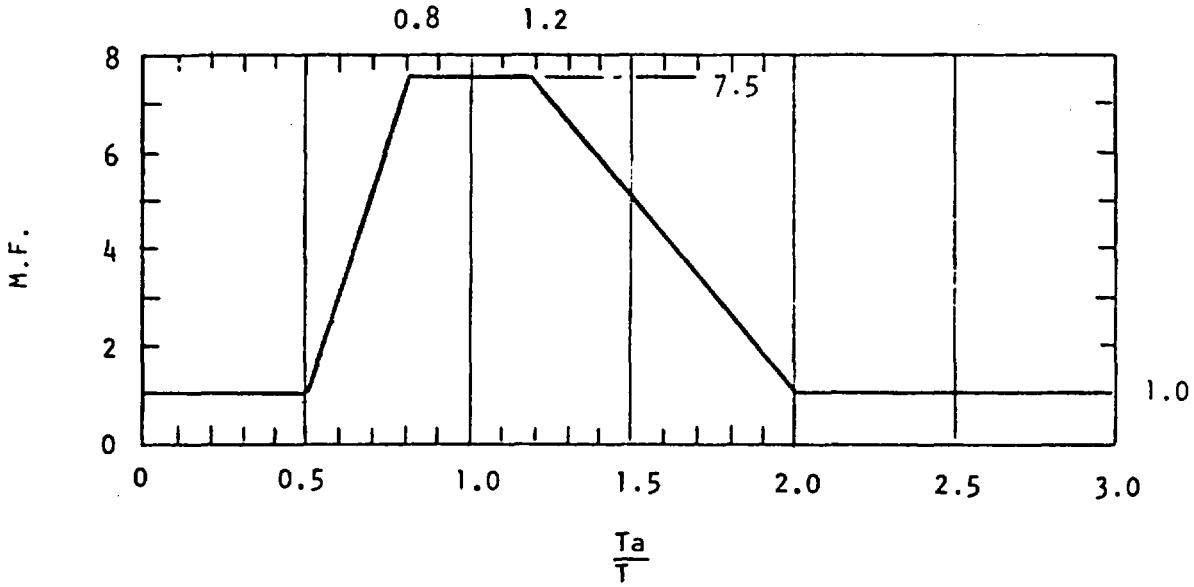

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Figure 6-1. Design M.F. versus period ratio.

(a) For each modal period, T_m, develop a plot of T_a versus S_{fa} from the standardized magnification curve in figure 6-1 using the following relationships:

$$T_a/T_m = T_a/T \qquad \text{(eq 6-3)}$$

$$S_{fa} = a_{xm}\ (\text{M.F.}) \qquad \text{(eq 6-4)}$$

Table 6-1 illustrates the tabulation of the pertinent data required for such a plot for an example where T_m = 2.0 sec and a_{xm} = 0.12g.

(b) Draw a horizontal line intersecting the ordinate at $S_{fa} = (a_x)_{max}$, where $(a_x)_{max}$ is the maximum floor acceleration from paragraph 6-3b. This line establishes the lower limit for S_{fa}.

(c) The floor response spectrum is defined by the envelope of the curves of paragraph *(a)* above and the lower limit established by paragraph *(b)*. An example of the procedure is illustrated by figure 6-2 from the data in tables 6-1 and 6-2. The example is for the roof of an assumed building. At other story levels of this building, the corresponding S_{fa} values will be proportional to the modal accelerations at those levels.

d. Maximum interstory drifts. The maximum lateral relative displacement between adjacent stories caused by EQ-I will be determined from the combined modal interstory drifts in accordance with chapter 4. Design example E-1 shows a method of determining the interstory drifts for each mode and the combined SRSS values are shown in table 5-3.

e. Design requirements.

(1) Rigid and rigidly mounted equipment or appendages (e.g., Ta < 0.05 sec) will be designed to resist the forces due to the maximum floor acceleration in accordance with the equation 6-5:

$$F_p = (a_x)_{max} W_p \qquad \text{(eq 6-5)}$$

where $(a_x)_{max}$ is determined from paragraph 6-3*b* and W_p is the effective weight of the equipment or appendage.

(2) The flexible or flexibly mounted equipment or appendages that can be represented as SDOF systems will be designed to resist the forces due to the appropriate floor spectral acceleration in accordance with equation 6-6:

$$F_p = S_{fax} W_p \qquad \text{(eq 6-6)}$$

where S_{fax} is the design spectral acceleration, S_{fa}, at floor x as defined in paragraph 6-3*c* for period T_a of the equipment or architectural appendage.

(3) Multi-mode systems will be designed by a modal analysis procedure similar to the procedure used for buildings in chapter 4, except that the floor response spectrum of paragraph 6-3*c* will be used in lieu of ground motion response spectrum.

(4) Nonstructural elements that are rigidly attached to two parts of the building that can move relative to each other will be designed to take the resulting deformations determined in paragraph 6-3*d*.

Table 6-1. Example of a response amplification curve for the building's fundamental mode of vibration.

Example:

Given: First mode period of vibration of building, T_m = 2.00 sec.
Maximum floor acceleration for first mode, a_{xm} = 0.12 g.

Find: S_{fa} values for response amplification curve for the first mode of building vibration.

Procedure: T_a/T and M.F. values are from figure 6-1.
T_a is obtained from equation 6-3.
S_{fa} is obtained from equation 6-4.

$T_m = 2.0, \quad a_m = 0.12$

T_a/T	0	0.5	0.8	1.2	2.0	3.0	fig. 6-1
M.F.	1.0	1.0	7.5	7.5	1.0	1.0	fig. 6-1
T_a	0	1.0	1.6	2.4	4.0	6.0	eq. 6-3
S_{fa}	0.12	0.12	0.90	0.90	0.12	0.12	eq. 6-4

$$T_a = T_m\ (T_a/T) = 2.0\ (T_a/T)$$
$$S_{fa} = a_{xm}\ (\text{M.F.}) = 0.12\ (\text{M.F.})$$

Table 6-2. Data for the floor (roof) response spectrum example of figure 6-2.

Example of Figure 6-2.

	Mode 1	Mode 2	Mode 3	SRSS
T_m (building), sec	2.00	0.61	0.36	
S_{am}, g	0.089	0.29	0.38	
PF_{xm} (x = roof)	1.30	0.45	0.22	
a_{xm}, g	0.12	0.13	0.08	0.19
S_{fa} (MF = 7.5)	0.90	0.98	0.60	

T_m from building analysis (chapter 4)

S_{am} from response spectrum for building period T_m

PF_{xm} from building analysis (eq.4-1)

$a_{xm} = PF_{xm}S_{am}$ is model story acceleration at level x for mode m (eq.6-1)

$S_{fa} = a_{xm}(M.F.)$ for maximum values at M.F. = 7.5 (eq.6-4). See Table 6-1 for other values on the amplification curve for mode 1

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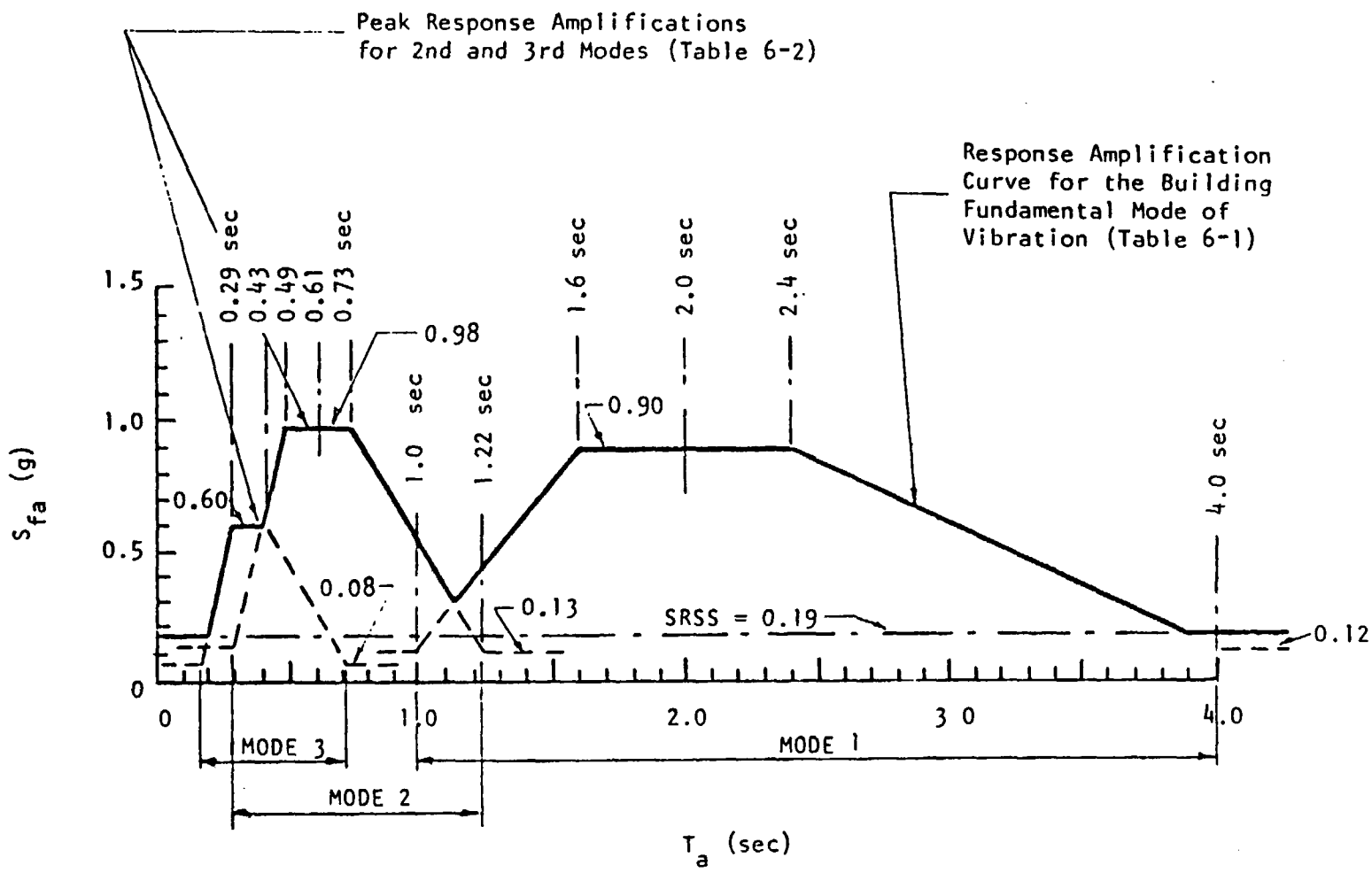

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Figure 6-2. Sample roof response spectrum.

6-4. EQ-II provisions.

The elements in or on the structure will be analyzed for their performance to the forces and distortions caused by response of the structure to earthquake motions that exceed the demands of EQ-I, up to and including the demands of EQ-II, in accordance with the criteria prescribed in this paragraph.

a. Method of analysis. The total design force representing earthquake effects will be determined from the maximum floor (or roof) accelerations of the building and from design floor (or roof) response spectra in the same manner as presented in paragraph 6-3, except as modified below.

b. Maximum floor acceleration. The maximum floor accelerations will be determined as prescribed in paragraph 6-3*b* for two conditions: (1) the maximum elastic capacity of the structure; and (2) the post-yield response of the structure caused by EQ-II criteria. The condition that results in the largest accelerations will govern the design of the nonstructural elements. If the elastic capacity of the structure exceeds the demands of EQ-II, the elastic response to EQ-II will govern the design.

(1) *Condition 1.* If the elastic capacity of the structure is approximately equal to the demands of EQ-I, *Condition 1* is automatically satisfied by the provisions of paragraph 6-3. However, if the elastic capacity of the structure significantly exceeds the demands of EQ-I, maximum story accelerations greater than those determined from an inelastic analysis (Condition 2) can result. The maximum floor accelerations determined from the elastic capacity of the structure are equal to the values obtained from the provisions of paragraph 6-3*b* multiplied by the ratio of the *elastic capacity of the structure* to the *demands of EQ-I.* This ratio is designated as the *elastic capacity ratio* (not to be confused with the *inelastic demand ratio*), and is determined from the provisions included in paragraph 4-3. Guidelines are provided in paragraph 5-4*e*(2).

(2) *Condition 2.* The maximum floor accelerations for the post-yield response caused by EQ-II will be determined from the combined modal story accelerations conforming to the provisions of paragraph 4-4.

c. Design floor response spectrum. The procedure outlined in paragraph 6-3*c* will be modified to determine the floor response spectra for two conditions: (1) the maximum elastic capacity of the structure; and (2) the post-yield response of structure caused by EQ-II criteria.

(1) *Condition 1.* If the elastic capacity of the structure significantly exceeds the demands of EQ-I, the amplitudes of the floor response spectrum calculated in accordance with paragraph 6-3*c* will be multiplied by the *elastic capacity ratio*, as defined in paragraph 6-4*b*(1).

(2) *Condition 2.* The magnification factors associated with post-yield response of the structure will tend to be less than those associated with linear-elastic response of the structure. Thus, the procedure outlined in paragraph 6-3*c* is modified by use of the magnification factor curve shown in figure 6-3.

d. Maximum interstory drifts. The maximum lateral relative displacement between adjacent stories caused by EQ-II will be determined from the combined modal interstory drifts in accordance with chapter 4.

e. Design requirements. The requirements prescribed in paragraph 6-3*e* will apply, except that references to paragraphs 6-3*b*, *c*, and *d* will be changed to paragraphs 6-4*b*, *c*, and *d*.

6-5. Architectural elements.

Architectural elements must: (1) safely resist horizontal forces equal to the design accelerations times their own weight; and (2) be capable of conforming (accommodating) to the lateral deflections that they will be subjected to during lateral deformation of the building in which they are located. The design of architectural elements will conform to the provisions of this chapter and the applicable portions of the Basic Design Manual, chapter 9. Architectural elements that are part of essential systems will also conform to the provisions of paragraph 6-7.

6-6. Mechanical and electrical elements.

a. General. The anchorage and support of mechanical and electrical equipment will be designed in accordance with the provisions of this chapter and the applicable portions of the Basic Design Manual, chapter 10.

b. Equipment certification. Manufacturers of essential mechanical and electrical equipment will provide certification, based on experimental or approved analysis, that the equipment will not sustain damage that may impair its function if it is subjected to the postulated motion.

c. Essential systems. Mechanical and electrical equipment that is part of essential systems will conform to the provisions of this chapter, paragraph 6-7.

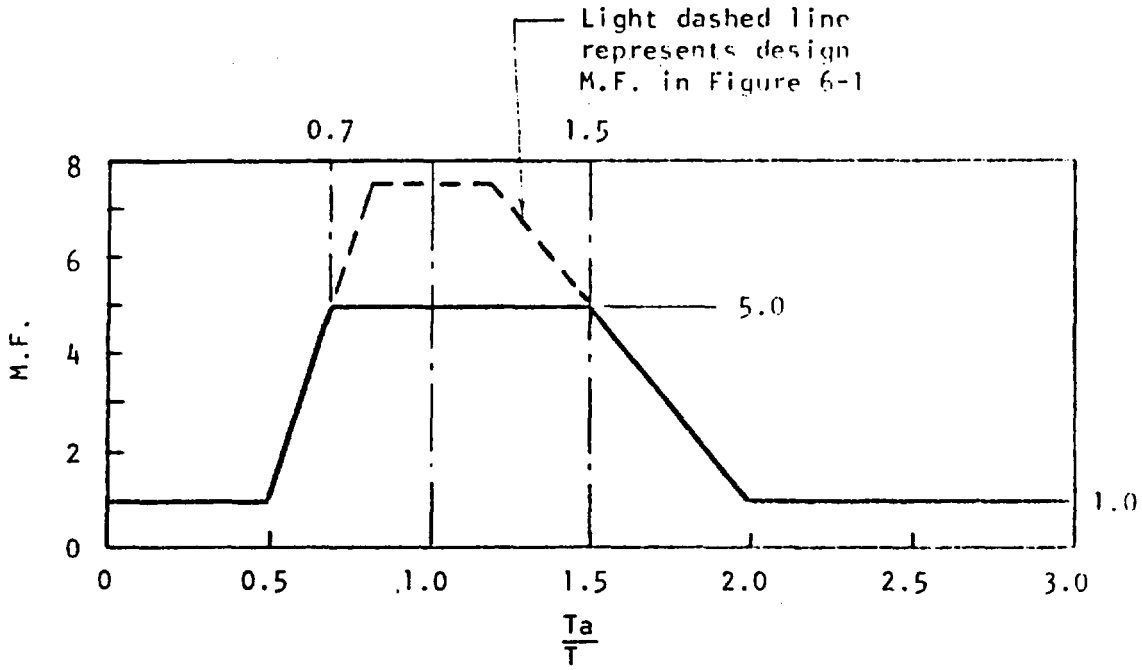

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Figure 6-3. Post-yield M.F. curve.

6-7. Essential systems.

Critical facilities are by definition those facilities that must provide needed services following a major disaster such as an earthquake. This implies that not only must the structure survive a major seismic disturbance, but also that essential nonstructural systems include all elements that are needed for the performance of emergency services or that may, by their failure, cause bodily injury or impair the performance of services. These systems are outlined in table 6-3.

a. Fire protection. The fire protection system, including both fire-fighting equipment and means of egress, is an important nonstructural system, since post-earthquake fires often cause more damage and injury than the earthquake itself.

(1) Special attention must be given to the protection of fire-fighting equipment. The sprinkler system piping shall be braced in accordance with NFPA No. 13 (Basic Design Manual, chap 10, para 10-7*a*), and fire pumps shall be governed by NFPA No. 20. Mounting brackets for hung and free-standing fire extinguishers shall be designed to prevent release of the extinguisher caused by horizontal or vertical earthquake motions.

(2) Exitways must not become blocked after an earthquake. Walls, ceilings, and lighting in exit corridors and all approaches to exits must be designed with extreme care. Door frames must be rigid enough to withstand imposed lateral forces and be detailed to allow wall movement. Stairs must also be given special attention so that they will not fail due to lateral loads or structural deformations (Basic Design Manual, chap 4, para 4-7*d*). Any glass used within exitways shall be tempered and its frame shall be designed to allow deformations (Basic Design Manual, chap 9, para 9-4*e*). Nonessential elements, such as display cases, should not be located in or near exitways where they may hinder egress.

b. Protection against hazardous materials. Hazardous materials pose a threat to post-earthquake operational capability of the facility as well as human safety.

(1) The types and quantities of hazardous materials present should be identified in the preliminary design phase.

(2) All distribution and storage systems for such materials shall be designed with extreme care. Fuel lines, bottles of laboratory chemicals, lead storage safes for radioactive materials, liquid oxygen storage tanks, and similar containers must be braced and protected from damage cause by movement or failure of adjacent elements. Seismic-activated shut-off valves shall be used at appropriate locations on supply

Table 6-3. Essential nonstructural systems.

FIRE PROTECTION
- Sprinkler and Standpipe Systems
  - Risers, Mains, and Branch Lines
  - Valves and Sprinklers
  - Support Hangers, Bracing and Clamps
  - Fire Pumps
  - Water Tanks
- Extinguishers
- Exits
  - Stairs
  - Doors
  - Corridors

HAZARDOUS MATERIALS
- Storage Tanks, Bottles, Cylinders, and Pipes Containing:
  - Natural Gas
  - O_2
  - N_2O
  - Anesthetic Gases
  - Chemicals
  - Radioactive Materials
  - Fuels

EMERGENCY POWER
- Substation
  - Transformer
  - Controls
  - Switchgear
- Engine-Generator
  - Engine Set
  - Fuel Tank and Piping
  - Cooling System
  - Exhaust System
- Batteries and Racks
- Switchboards and Panelboards
- Motor Controllers and Control Centers

COMMUNICATIONS
- Alarms
- Telephone
- Radio
- PA System
- Paging System
- Intercom System
- Nurses' Call

TRANSPORT
- Elevators/Dumbwaiters
- Cabs
- Rails
- Counterweights
- Motors
- Generators
- Controls

MECHANICAL
- Water System
  - Tanks
  - Heaters
  - Pumps
  - Risers, Mains, and Branch Lines
  - Valves
- Sanitary System
  - Soil Stacks and Branch Lines
  - Vent Stacks and Branch Lines
  - Storm Leaders (if connected to sanitary)
  - Building Drains and Sewers
- HVAC Systems
  - Boilers
  - Pipes, Ducts and Hangers
  - Converters
  - Heat Exchangers
  - Compressors
  - Condensers
  - Chillers
  - Air Handling Units, Blowers, and Fans
  - Cooling Towers
  - Furnaces
  - Chimneys
- Miscellaneous
  - Vacuum Pump and Piping
  - Refrigeration and Medical Compressors
  - Kitchen Equipment
  - Laundry Equipment
  - Maintenance and Repair Supplies
  - Cleaning Supplies

ARCHITECTURAL
- Exterior Walls, Panels, and Glazing
- Interior Partitions and Facing Materials
- Ceilings
- Light Fixtures
- Horizontal and Vertical Projections
- Ornamentation
- Storage Units
- Essential or Potentially Hazardous Furnishings
- Computer Floors

ESSENTIAL SUPPORT
(to be defined for each type of facility)

OTHERS
- Elements in Proximity of Critical Equipment
- Expensive Equipment
- Computer Equipment

lines for natural gas and other hazardous materials.

c. Emergency power and circuits. An emergency system consisting of separate emergency circuits and an alternate on-site power source shall be provided.

(1) For health care facilities, this system shall be governed by the provisions of NFPA No. 76-A and TM 5-838-2. For all other critical facilities, the provisions of NFPA No. 70, Article 700, and this section shall apply. The requirements for the bracing of elements of the mechanical system, given in paragraph 6-7*f*, shall also apply to elements of the emergency power system.

(2) Emergency circuits shall consist of separate circuits serving lighting and equipment that is essential for life-safety and the performance of post-earthquake operations. The extent of this system shall be determined for each facility during the preliminary design phase. In general, it will include:

—Illumination of means of egress.
—Alarm and alerting devices.
—Emergency communications systems.
—Illumination of generator set location.
—Task illumination for essential services.
—Essential equipment.
—At least one elevator, plus ventilation, communications, and lighting for all other elevators.

(3) The emergency circuits shall be served by the normal power source of the electrical system and, upon failure of the normal source, by at least one alternate source. The main feeders for the emergency circuits shall be physically separated from normal wiring to prevent their simultaneous destruction.

(4) The normal power source preferably should consist of two separate full-capacity services, connected in such a manner that one will automatically pick up the load upon loss of the other. Upon failure of both sources, the load shall be transferred to the alternate power source. No power source shall have a capacity less than that required by the emergency circuit system. Automatic transfer devices shall be located in protected places and adequately anchored.

(5) The principal alternate power source shall be a generator set driven by an acceptable prime mover located on the premises, preferably at ground level. If possible, a generator with an integral radiator cooling system should be used. If an auxiliary cooling system is necessary, the cooling tower or remote radiator should be installed at grade level. All equipment and piping shall be braced.

(6) An on-site fuel supply sufficient for the maximum estimated emergency period shall be provided. The fuel storage tank should be located underground and properly anchored. Flex loops should be used in fuel lines between the tank and building and at the connection to the generator. Malleable fittings and valves should also be used.

(7) Conductors should cross earthquake or expansion joints only at lower levels and with adequate provision for differential movement. Separate grounds for branch circuits crossing these joints should be provided.

d. Communications. The post-earthquake communication requirements of a facility must be defined during the preliminary design phase.

(1) An internal communication system that can operate independently of the telephone system and normal power supply may be required. An external communication system capable of contact with community and state emergency services as well as mobile units (such as police cars or ambulances) shall be provided.

(2) Emergency communication equipment must be located in a nonvulnerable portion of the facility, preferably the lower levels, and must be designed and mounted to resist seismic motion.

e. Transport. All elevators, shafts, and accessories shall be designed to resist the lateral-force requirements. At least one elevator, plus the ventilation, communication, and lighting for all elevators, shall be connected to the emergency power system.

(1) For traction elevators (Basic Design Manual, fig 10-3, chap 10, para 10-10), insure that counterweights cannot become derailed by strengthening their guide rails using additional or stronger rail brackets and installing safety shoes on the counterweight assembly. Guide rails for cars are normally designed for large lateral forces, but it may be necessary to install spacers between back-to-back rails at midpoints between spreader beams. Use of loose traveling cables in hoistways should be avoided if possible, or sheave guards should be used to contain the cables. Motor generators, motor drives, and traction machines shall meet the criteria given for mechanical systems in paragraph 6-7*f*.

(2) Hydraulic elevator equipment should be properly secured and splash-proof oil tanks should be used.

(3) All elevator door frames must accommodate predicted interstory movement to prevent jamming. Selector and controller panels and their components must be adaquately secured.

Heat-sensitive call and floor buttons shall not be used.

f. Mechanical. The seismic design of mechanical systems requires attention not only to the various system components, but also to their interfaces and linkages, since failure most often occurrs at these points during an earthquake. Safe and easy access to all components shall be provided to facilitate maintenance and repair of the mechanical systems. A program of periodic testing and inspection must also be established.

(1) *Water system.* Two independent connections to the exterior water supply are required (Basic Design Manual, chap 12, paras 12-5*b* and 12-6*c*). Water testing equipment for monitoring the normal water supply and standby chlorination equipment for disinfection of the water may also be necessary. A separate water storage facility containing a water supply adequate for the post-earthquake emergency period shall be provided. The water distribution system within the facility shall be designed to conserve the emergency supply through the use of shutoff valves for branches to nonessential fixtures.

(2) *Sanitary system.* For high-occupancy facilities such as hospitals, an emergency sewage-holding facility shall provide for temporary retention of sewage discharged during a period of four days. Sewer and vent lines must be protected from damage to structural deformation or movement of adjacent elements.

(3) *Heating, ventilation, and air conditioning (HVAC) systems.* Critical heating, ventilation, and air conditioning requirements will vary according to the facility and location. Means of closing off nonessential portions of the HVAC systems shall be provided and special attention shall be given to design of the portions that must remain functional following an earthquake. All HVAC equipment, piping, and ducts shall be designed and braced according to the requirements of this section.

(4) *Equipment with vibration isolation.* Much of the damage to mechanical systems that occurs during earthquakes is incurred by equipment with vibration isolation, such as helical springs, air cushions, rubber-in-shear mounts, or fiber-in-shear mounts. All vibration isolation systems shall be capable of resisting the same horizontal force per inch of travel that is resisted the same horizontal force per inch of travel that is resisted vertically. The systems shall be attached to both the floor slab or supporting structural member and the supported equipment. Restraining devices shall be provided to limit all horizontal and vertical motion, prevent overturning, and inhibit resonance.

(5) *Equipment without vibratory isolation.* All equipment shall be bolted or rigidly attached by other means to the floor slab or supporting structural member. Suspended equipment shall be adequately braced against movement in all directions or mounted tightly against a structural member.

(6) *Piping.* Pipes with an inside diameter of 2½ inches or larger, as well as all fuel gas pipes, acid waste pipes, and pipes within boiler and equipment rooms, must be braced (Basic Design Manual, chap 10, para 10-7). Maximum spacing for transverse bracing is 40 feet on center, and for longitudinal bracing is 80 feet on center. Transverse bracing for one pipe section may also act as longitudinal bracing for a perpendicular pipe section if the brace is within 24 inches of the connecting elbow or tee. Branch lines may not be considered bracing for the main line. Each vertical riser shall be supported at a point or points above its center of gravity. Also, laternal guides should be provided at the top and bottom of the riser and at intermediate points not to exceed 40 feet on center. Care must be taken in routing piping. Piping should cross building seismic or expansion joints only in the lower levels of the facility. Flexible joints and damage control valves must be provided where pipes pass through such joints, where rigidly supported pipes connect to equipment on resilient mountings, and where pipes enter and exit the facility. Piping shall be designed to prevent damage from movement of the structural system. Pipes within a partition should be anchored to the same structural member as the partition. A rigid piping system should not be fastened to dissimilar structural elements or building parts, since their responses to earthquake motion may differ. Appropriately located zone or damage control valves are required for unbraced pipes to limit system outages in case of failure. Malleable rather than cast-iron fittings and valves shall be specified. Pipe sleeves large enough to allow anticipated differential movement shall be provided where pipes pass through floors or walls. A 6-inch lateral clearance is required between unbraced piping and adjacent piping, ducts, hangers, and other elements.

(7) *Ducts.* Lateral bracing must be provided for all ducts with a perimeter greater than 120 inches and for all ducts in boiler and equipment rooms. Maximum allowable spacing for transverse bracing is 30 feet on center. Transverse bracing shall also be installed at each turn in the duct and at the end of a duct run. Longitudinal bracing shall occur at 60 feet, maxi-

mum spacing. Transverse bracing for a duct section may also act as longitudinal bracing for a duct section perpendicular to it, if the bracing is installed within 4 feet of the intersection of the ducts and if the bracing is sized for the larger duct. No bracing is required if attachment is made from the top of the duct directly to the supporting structural member; if the distance between the top of the duct and the member is 12 inches or less; and if a 6-inch laternal clearance is provided between adjacent piping, ducts, hangers, or other elements. Walls, including nonbearing gupsum board partitions, may be considered transverse bracing for ducts that pass through them. Ducts may be grouped in a laternal bracing frame if the frame is sized and designed for the total group. Diffusers, registers, and grilles shall be positively attached to the ductwork. If ducts are flexible, positive connections must also be made to the ceiling, wall, or floor system.

(8) *Site utilities.* All on-site utility lines shall be designed to minimize disruption by earthquakes (Basic Design Manual, chap 12). Natural gas lines shall be equipped with earthquake-sensitive automatic shut-off valves in addition to manual shut-off valves. Dual supply systems shall be separated as much as possible to limit the chance of simultaneous disruption of both supplies.

g. Architectural. Architectural systems, particularly walls, ceilings, and floors, shall be detailed to maintain the integrity of building seismic or expansion joints. If these elements are continued without break over the joints, they may act to tie the building sections together, thus changing the response to seismic motion.

(1) Exterior wall systems. Allowance of interstory drift is extremely important in detailing exterior wall systems, including glazing. The calculated story drifts or ½-inch, whichever is greater, shall be used in design. Special attention must also be given to the method of anchoring exterior wall panels (Basic Design Manual, chap 9, para 9-4*b*). Care should be taken to prevent corrosion from reducing the strength of the connections. Stone panels with metal anchors are particularly susceptible to damage during earthquakes and are not recommended for use on buildings with predicted interstory drift greater than L/300, where L is the height between floors in the same units as the interstory drift.

(2) Interior particions and facing materials. Interior walls and partitions that are not shear walls be designed to allow for interstory drift (Basic Design Manual, chap 9, para 9-4*a*). Damage can be prevented by anchoring each partition along one edge to a single structural member and allowing movement at the other edges. Brittle facing materials, such as ceramic tile or glazing masonry, suffer extensive damage during earthquakes and should be used only when necessary.

(3) *Ceilings.* Flexible ceiling systems, such as exposed tee bar, concealed spline, or luminous systems, shall not be used unless the following provisions are made (Basic Design Manual, chap 9, para 9-4*a*). They shall be braced at regular intervals against lateral and vertical motion, cross runners shall be securely fastened to main runners with locking clips or wire ties, the ceiling shall be isolated from walls and partitions by a soffit or edge angle wide enough to allow movement, and hangers shall be provided at the perimeter so that the wall does not support the ceiling. Gypsum board and lath and plaster ceilings are more rigid and, therefore, tend to be more earthquake-resistant. However, bracing shall be provided at regular intervals against vertical movement and at the perimeter against lateral movement. Gypsum board ceilings shall be reinforced at nail points with steel nailing strips. Allowance shall be made at ceiling openings for movement of diffusers, sprinklers, and other equipment connected to rigid mechanical systems.

(4) *Light fixtures.* No light fixture shall be installed without positive attachment to the supporting element by means of bolts or locking devices (Basic Design Manual, chap 10, para 10-6). Fixture accessories, such as louvers, diffusers, and lenses, shall also have lock or screw attachments. Recessed and lay-in fixtures shall be supported by and secured to the main runners of the ceiling support system, not furring cross runners or nailing bars. Where the positions of the main runners do not coincide with the lighting configuration, auxiliary support members of equal strength shall be provided. Secondary supports consisting of two wires, each capable of supporting four times the fixture weight, shall be placed at the diagonal corners of each fixture and attached to the structural system. Pendant fixture should not be used, since they are highly susceptible to damage from earthquakes and can also inflict considerable damage on ceilings because of their large response motions. If high ceilings necessitate the use of a lower lighting system, a supporting grid designed and braced according to ceiling requirements should be used.

(5) *Horizontal and vertical projections.* All balconies, overhangs, parapets, and other

projecting elements shall be braced against lateral and vertical movement. Special attention must be given to elements that might be affected by deflection of the cantilevers.

(6) *Storage units.* Overturning or sliding storage units can cause personal injury in addition to disorder. Units shall be anchored and braced to resist lateral and uplifting forces. Parallel rows of racks, shelves, or file cabinets should have rigid bracing across the tops of the units to stabilize the entire configuration. File drawers and cabinets shall have latches that will prevent their opening and subsequent spilling of contents during the earthquake. Shelves shall be provided with face bars to prevent spilling of contents.

(7) *Computer floors.* Computer floors shall be adequately braced and drop-in panels detailed to prevent displacement during an earthquake.

(8) *Essential or potentially hazardous furnishings.* Furnishings that are essential for post-earthquake operation or that might pose a serious hazard either to persons or to essential systems shall be adequately braced.

h. Essential support. For each facility, the equipment essential to the performance of post-earthquake services shall be identified during the early stages of design. Guidance for medical systems in health care facilities is given in TM 5-838-2. In general, the following should be done to insure that an element should survive an earthquake in operable condition:

—Check the adequacy of the element to resist its inertial force.

—Brace it in a manner convenient for daily use.

—Insure that it will not be damaged by structural deformation or the failure or movement of adjacent elements.

i. Others. Throughout the design process, care should be taken to identify elements that merit special seismic considerations. Such elements may be unusually expensive equipment, computer equipment, or elements in close proximity or critical equipment. The general guidelines given for essential support equipment shall also apply to these elements.

CHAPTER 7
STRUCTURES OTHER THAN BUILDINGS

7-1. Introduction.

This chapter prescribes the seismic design criteria for structures other than buildings that must remain intact or functional after a major seismic disturbance. This includes structures, independent of buildings, that are located on the ground. The criteria and design standards of this chapter provide a dynamic analysis approach to the seismic design of structures other than buildings that is used in lieu of the lateral static force procedure of the Basic Design Manual, chapter 11.

7-2. General requirements.

Structures other than buildings will be designed in accordance with the general requirements and the design and analysis provisions of chapter 4 of this manual, with the exceptions noted in this chapter.

a. Damping. Damping values for structures other than buildings will be lower than the values allowed for buildings of similar lateral-force-resisting systems. Thus, damping values of table 4-1 will be modified, as shown in table 7-1. The lower damping will result in larger lateral forces and distortions. This is justified because of the general lack of partitions and other nonstructural elements that contribute to the energy-absorbing characteristics of buildings. Where it can be demonstrated that these energy-absorbing characteristics are present, the higher damping values may be used.

b. Structural component load effects. For structures other than buildings, the percentages of exceedance to strength requirements of paragraph 4-3e(1) are not permitted unless a prescribed degree of redundancy can be demonstrated. This is justified on the basis that these structures generally do not have the multiplicity of structural and nonstructural resisting elements characteristic of most buildings. Exceptions: redundancy can be assumed for *ductile structures* for the following:

(1) When the structure consists of two column lines of lateral-force-resistance in each principal horizontal direction of motion and there are a minimum of four vertical elements in each column line designated to resist the horizontal forces, flexural strength requirements may be exceeded by a value up to 15 percent.

(2) When the structure consists of four column lines with a minimum of four vertical elements each to resist horizontal forces in each direction, or where there are two column lines with a minimum of eight vertical elements each to resist horizontal forces in each direction, flexural strength requirements may be exceeded by a value up to 25 percent.

7-3. Elevated tanks and other inverted pendulum structures.

Structures that represent inverted pendulums, such as an elevated tank supported by a tower structure that is light in weight relative to the tank and contents, will use the lower damping values of paragraph 7-2*a* and will not be permitted the exception of paragraph 7-2*b*. The value for W will include the effective weight of the contents. The accidental torsion will be computed as for buildings. The structure will be analyzed for earthquake forces in any horizontal direction.

Table 7-1. Damping values for structures other than buildings.

Buildings	Structures Other Than Buildings
0.03	0.015
0.05	0.02
0.07	0.05
0.10	0.07

a. Elevated tanks on cross-braced columns. The provisions of the Basic Design Manual, paragraph 11-3b, generally apply.

b. Hydrodynamic effects. The provisions of the Basic Design Manual, paragraph 11-3*b*, generally apply. The procedure for analyzing a two-degree-of-freedom system, taking into account the effects of the sloshing liquid, may require a parameter study representing various levels of liquid containment. The response characteristics will vary with the percentages of liquid in the elevated tank. Thus, the critical condition may not always occur with a full tank. For example, the analysis might consider the condition of the tank three-fourths full, one-half full, and one-fourth full.

c. Elevated tanks, pedestal-type. Pedestal-type elevated tanks will not be permitted in zones of high seismicity (i.e., Basic Design Manual seismic zones 3 and 4).

7-4. Vertical tanks (on ground).

The design criteria for vertical storage tanks on the ground will follow the general procedures prescribed in the Basic Design manual, paragraph 11-4, except that response spectra will be substituted for coefficients ZIKCS.

a. Rigidly contained liquid. For tanks in which the liquid is rigidly contained (i.e., sloshing prevented), for tanks holding highly viscous materials, and for pressure tanks, the design forces will be based on the peak spectral acceleration on the design response spectrum unless a lower value can be substantiated by a properly calculated period of vibration for the tank structure.

b. Hydrodynamic effects. For tanks where the liquid is not rigidly contained, the hydrodynamic effects of the sloshing liquid may be considered. The rigid body forces will be determined from the peak spectral acceleration on the design response spectrum, and the sloshing liquid forces will be determined by the spectral acceleration consistent with the sloshing period (Basic Design Manual formula 11-4, para 11-3a(2)(b)).

7-5. Horizontal tanks (on ground).

The provisions of the Basic Design Manual, paragraph 11-5, generally apply. Response spectra may be substituted for base shear coefficients where applicable.

7-6. Retaining walls.

The provisions of the Basic Design Manual, paragraph 11-6, generally apply. Response spectra may be substituted for base shear coefficients where applicable.

7-7. Buried structures.

The provisions of the Basic Design Manual, paragraph 11-7, generally apply. Response spectra may be substituted for base shear coefficients where applicable.

APPENDIX A
SYMBOLS AND NOTATIONS

A-1. Symbols and notations

Symbols and notations are divided into two sections: ground motion chap 3 and app C) and buildings (chaps 4, 5, 6, and 7).

A-2. Ground motion (chapter 3 and appendix C)

A	= peak ground acceleration in cm/sec^2
α	= intercept of the log-recurrence line
β	= slope of the log-recurrence line
EPA	= effective peak acceleration
EPV	= effective peak velocity
EQ-I	= seismic ground motion having 50-percent probability of exceedance in 50 years
EQ-II	= seismic ground motion having 10-percent probability of exceedance in 100 years
DAF	= dynamic amplification factor
m_b	= body wave magnitude
M_L or M	= Richter or local magnitude
M_s	= surface wave magnitude
M_o	= seismic moment
M_m	= seismic moment magnitude
PGA	= peak ground acceleration
PGV	= peak ground velocity
PGD	= peak ground displacement
R_e	= effective distance
R_E	= epicentral distance
R_H	= hypocentral distance
SD	= relative displacement response spectrum
SV	= relative velocity response spectrum
SA	= absolute acceleration response spectrum
I_o	= modified Mercalli intensity at the epicentral area
I or MMI	= modified Mercalli intensity at the site
S_a	= response spectrum value for pseudo-acceleration
S_v	= response spectrum value for pseudo-velocity
S_d	= response spectrum value for displacement
T_R	= return period in years
$\ddot{x}(t)$	= corrected accelerogram record of ground motion
$\dot{x}(t)$	= computed ground velocity record
$x(t)$	= computed ground displacement record
t	= time in seconds
ω	= circular natural frequency in radians per second
k	= stiffness
c	= viscous damping
m	= system mass
T	= structural period in seconds
V	= coefficient of variation

A-3. Buildings (chaps 4, 5, 6, and 7)

A_G	= an effective peak ground acceleration to define S_a at a response period, T = 0
a_{xm}	= story lateral acceleration at level x for mode m
$(a_x)_{max}$	= maximum acceleration at level x, including effects of modal combinations
C_{bm}	= modal base shear coefficient for mode m. Equivalent to ZIKCS coefficient in Basic Design Manual, equation 3-1
subscript C	= denotes a force in terms of *capacity*
D	= dead load
subscript D	= denotes a force in terms of *demand*
E	= earthquake load
EC	= elastic capacity to resist the seismic effects, from equations 4-6, 4-7, and 4-8
EQ-I	= earthquake that has a 50-percent probability of being exceeded in 50 years
EQ-II	= earthquake that has a 10-percent probability of being exceeded in 100 years
F_{xm}	= story lateral force at level x for mode m
g	= acceleration due to gravity
K	= stiffness of a system in terms of force required for a unit of lateral displacement ($K = F/\delta$) *Note:* not to be confused with the K used as a coefficient in the Basic Design Manual
K	= numerical coefficient as set forth in Basic Design Manual table 3-3
K*	= normalized stiffness of a system that is a function of the dynamic characteristics of the system
L	= live load
M	= mass of a system ($M = W/g$)

M^* = normalized mass of a system that is a function of the dynamic characteristics of the system

MDOF = Multi-degree-of-freedom system

M.F. = magnification factor to obtain floor response spectrum in equation 6-4

N = number of stories above the base to level n

n = the level that is uppermost in the main portion of the structure (generally the roof)

PF_{xm} = modal participation factor at level x for mode m, from equation 4-1

R_v = ratio of Basic Design Manual shear to modal analysis base shear, from equation 5-1

RSS = root-sum-squares, same as SRSS

S_a = spectral acceleration, as a ratio of the acceleration of gravity (g)

S_{am} = spectral acceleration for mode m

S_{dm} = spectral displacement for mode m

S_{fa} = spectral acceleration of a floor response spectrum

S_{fax} = spectral acceleration of floor response spectrum at level x

SDOF = single-degree-of-freedom system

SRSS = Square-root-of-the-sum-of-the-squares

S_1, S_2, S_3 = soil types for developing ATC-3-06 response spectra (NBS 510)

t = time in seconds

T_a = period of vibration of equipment or architectural appendage

T_m = period of vibration for mode m. T_1 designates the fundamental mode, T_2 designates the second mode, etc.

V_m = total lateral force for mode m

W = weight of a system or building

W_i/g = mass assigned to level i

W_p = weight of a portion of a structure, equipment, or architectural appendage

W_x = weight at or assigned to level x

α_m = modal base shear participation for mode m, from equation 4-2

β = damping as a percentage or ratio

δ = lateral displacement

δ_{xm} = lateral displacement at level x for mode m

Δ_{xm} = modal lateral interstory drifts for mode m within story x (e.g., the difference between δ_{xm} at story x = x + 1 and story x = x)

ϕ_{im} = amplitudes of mode m at levels i, from i = n to i = 1

ϕ_{xm} = amplitude of mode m at level x

θ = P-delta stability coefficient, as defined in paragraph 5-5d and ATC-3-06 (NBS 510)

APPENDIX B
REFERENCES

Government Publications.

Department of the Army.

TM 5–809–10 Seismic Design for Buildings
TM 5–838–2 Army Health Facility Design

Department of the Air Force.

AFM 88–3, Chapter 13 Seismic Design for Buildings

Department of the Navy.

NAVFAC P–355 Seismic Design for Buildings
NAVFAC P–355.1 Seismic Evaluation of Supports for Existing Electrical-Mechanical Equipment and Utilities

National Bureau of Standards (NBS).

National Technical Information Service, 5285 Port Royal Road, Springfield, VA, 22161

or

Superintendent of Documents, U.S. Government Printing Office, Washington, DC, 20402
Special Publication 510 (514 pages), Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC–3–06), 1978

Nongovernment Publications.

National Fire Protection Association, Inc. (NFPA) Batterymarch Park, Quincy, MA, 02269

NFPA No. 13, Sprinkler Systems

NFPA No. 20, Centrifugal Fire Pumps

NFPA No. 76–A

NFPA P. 70, Article 700

Stanford University, The John A. Blume Earthquake Engineering Center, Stanford, CA, 94305

Technical Report No. 36, Computer Programs for Seismic Hazard Analysis—A User Manual (STASHA), G. A. Guidi, 1979

American Concrete Institute (ACI), Box 19150, Redford Station, Detroit MI, 48219

ACI 318–77, Building Code Requirements for Reinforced Concrete

Portland Cement Association (PCA), Old Orchard Road, Skokie, IL, 60076

Advanced Engineering Bulletin No. 20, Biaxial and Uniaxial Capacity of Rectangular Columns, 1967

APPENDIX C
GROUND MOTION BACKGROUND DATA

C–1. Earthquake Source and Earthquake Size Definition.

The actual release of earthquake energy along the fault plane in the crust of the earth is a very complex phenomenon. All the physical processes that occur just before, during and after a seismic event are still not completely understood, and considerable research is going on to better describe this phenomenon. However for engineering purposes, the above complex phenomenon is idealized, and figure C–1 gives the resulting simplified model representation of the earthquake source.

a. Earthquake location. Epicenter and Hypocenter are the two terms most commonly used to describe the source location of an event. Even though most of the seismic energy is released as the fault ruptures and that a substantial volume of the earth's crust (along the fault plane) is involved, it is generally assumed that there exists a discrete point where the rupture initiates. This point where the initial rupture of the rocks within the earth's crust begins is called the *hypocenter.* The point directly above the hypocenter on the earth's crust is called the *epicenter.* In recent times (since the beginning of seismographs), the location of the hypocenter and hence the epicenter is made by means of instruments. Before the advent of the instruments, the epicenter was located by means of finding the region of intense shaking. It is quite often that the field epicenter (region of intense shaking) and the instrumentally located epicenter do not coincide. See figure 3–22.

b. Earthquake size. Various empirical relationships are available to relate the size of the event with the rupture length and fault slip. The fault rupture length is the length of the fault that actually breaks on the surface of the earth. The fault slip is the relative displacement of the two plates with respect to each other at the fault plane. Figure C–2 shows different types of fault slips. Again, empirical relationships are available to relate earthquake size with slip length. To define the size of an earthquake, Charles Richter developed a Richter Magnitude scale. This scale is intended to be a rating given to an earthquake event, independent of the location of observation. The size was determined by means of a standard Wood-Anderson seismometer, with natural period of 0.8 seconds. Richter defined the Magnitude as the logarithm to the base ten of the ratio of the maximum amplitude on a seismogram written by a Wood-Anderson seismometer at a distance of 100 kms (62 miles) from the epicenter and the standard amplitude of one thousandth of a millimeter. Tables were constructed empirically to reduce from any given distance to 100 kms. Since the scale is logarithmic, an increase of one step on the magnitude scale increases the amplitude scale by a factor of 10. (See fig. C–3).

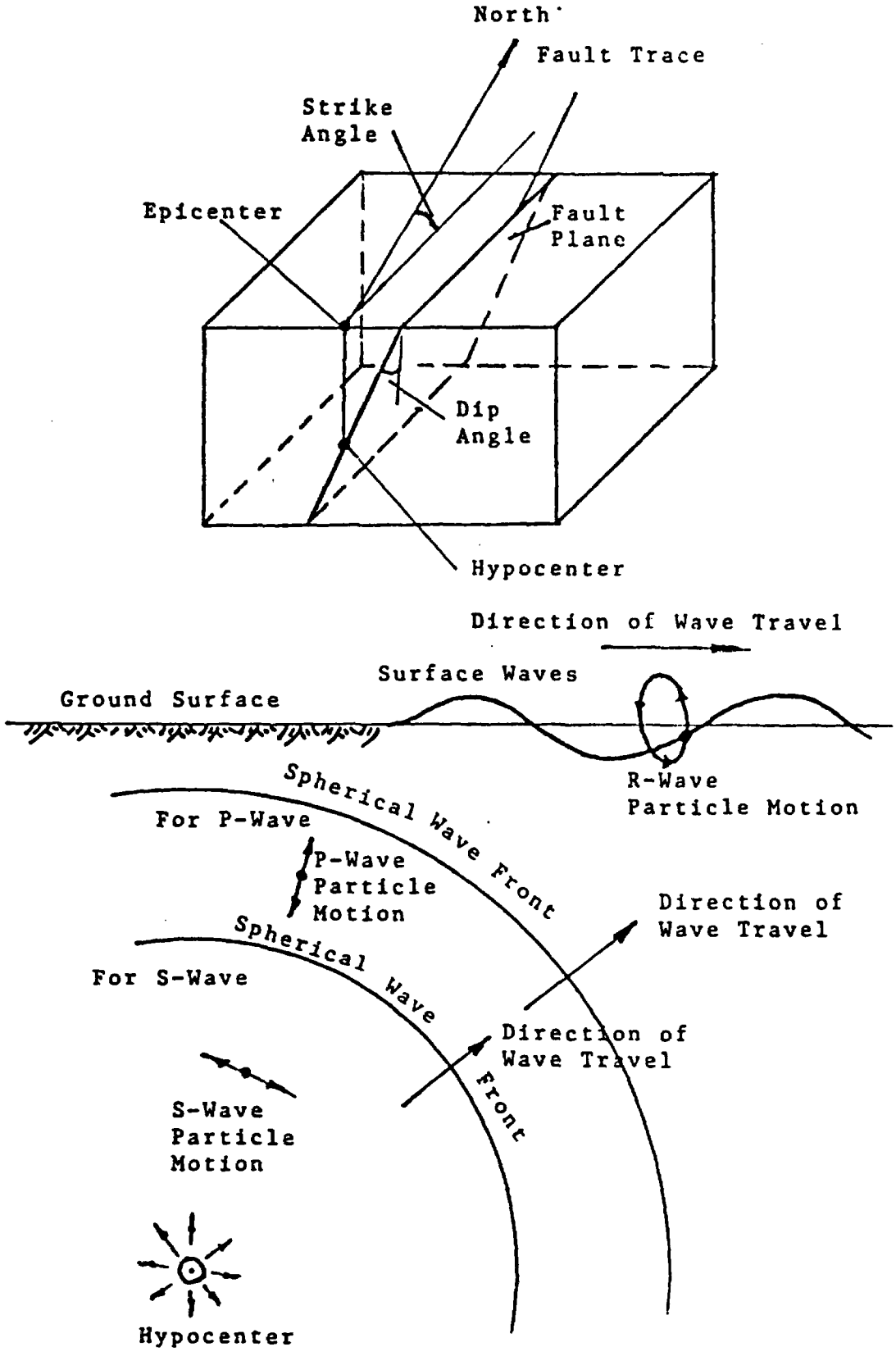
c. Other magnitude measures. In recent years, different types of instruments are used to obtain similar magnitude values which are referred to as local magnitude, M_L. The body wave magnitude m_b and the surface wave magnitude M_S are also used. In most studies, the local amplitude scale M_L is taken as a Richter magnitude. This assumption does introduce some errors in magnitude assignments. The local magnitude scale M_L can be related to the body wave magnitude m_b and the surface wave magnitude M_S by the following empirical relationships:

$$M_L = 1.34m_b - 1.71 \qquad \text{(eq C–1)}$$

$$M_L = 2.20[m_S - 3.80]^{1/2} + 2.97 \qquad \text{(eq C–2)}$$

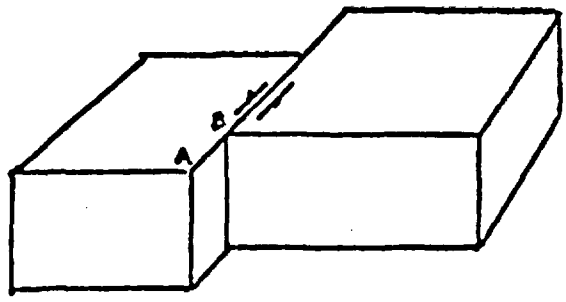
Surface-wave magnitude M_s is usually based on the amplitude of 20 second waves recorded at distances of thousands of kilometers. The reason for preferring local magnitude is that for large earthquakes the surface-wave magnitude may increase as the physical size of the source region increases without a corresponding increase in the amplitude of ground motion in the period range affecting normal structures. This is well illustrated by the Kern County earthquake of 1952 which had a surface wave magnitude of 7.7 and a local magnitude of 7.2 and by the San Francisco earthquake of 1906 with a surface-wave magnitude of 8.25 and a local magnitude of 7.2 or less. It is generally believed that the local magnitude scale saturates in the range of 7 to 7.5. The largest measured value to date is 7.2.

d. Seismic moment. As more is known about the earthquake source mechanism and about the size of earthquake events, it is becoming increasingly clear that the existing magnitude scales are extremely inadequate to describe the overall size or the energy content of earthquake events. To overcome this deficiency, seismolo-



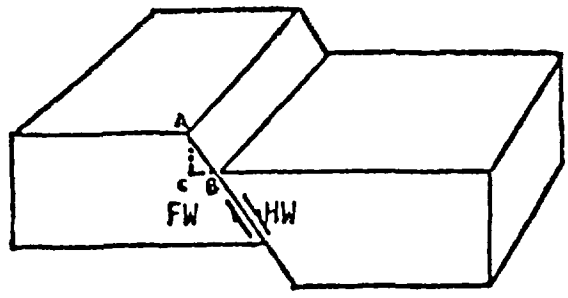
US Army Corps of Engineers

Figure C-1. Earthquake source model.

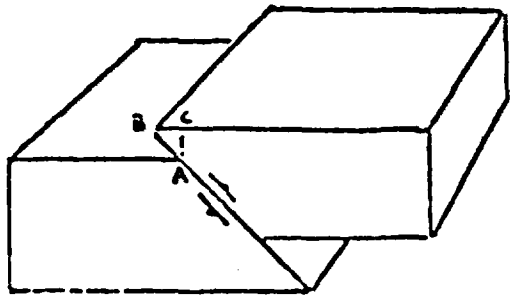


a. STRIKE-SLIP FAULT
(LEFT-SLIP FAULT)

AB=strike-slip= slip

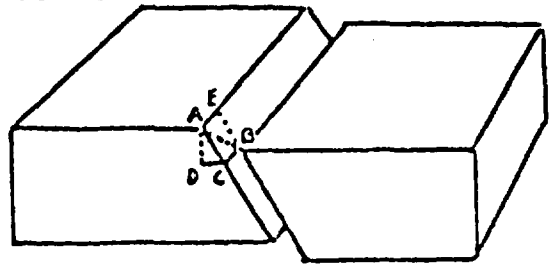


b. NORMAL-SLIP FAULT
AB = dip-slip= slip
AC = throw or vertical component
BC = heave or horiz. extension



c. REVERSE-SLIP (THRUST) FAULT

AB = reverse-slip = slip
AC = throw or vertical component
BC = heave = horiz. shortening

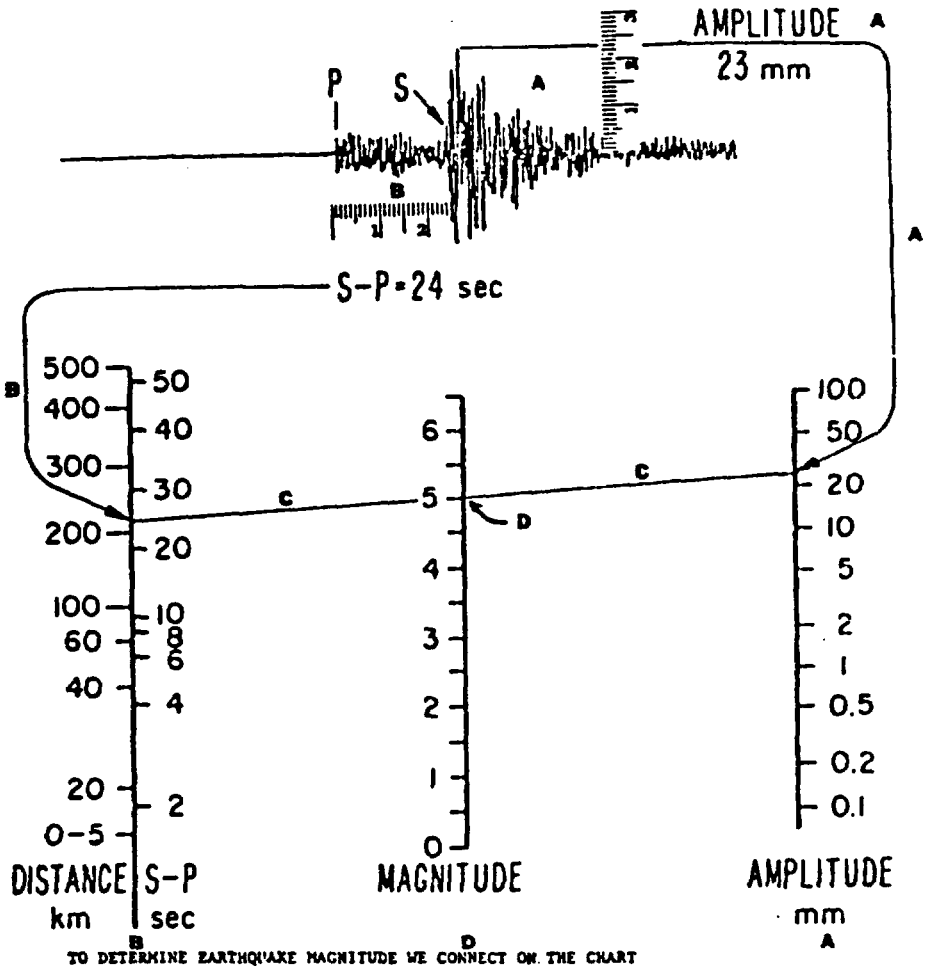


d. LEFT-OBLIQUE-SLIP FAULT

AB= oblique-slip = slip
AC = dip-slip component
AE = strike-slip component
AD = throw = vertical component
DC = heave = horiz. extension

US Army Corps of Engineers

Figure C-2. Types of fault slips.



TO DETERMINE EARTHQUAKE MAGNITUDE WE CONNECT ON THE CHART

A. THE MAXIMUM AMPLITUDE RECORDED BY A STANDARD SEISMOMETER, AND
B. THE DISTANCE SEISMOMETER FROM THE EPICENTER OF THE EARTHQUAKE (OR DIFFERENCE IN ARRIVAL TIMES OF P AND S WAVES) BY
C. A STRAIGHT LINE,
D. READ THE MAGNITUDE, ON CENTER SCALE.

Reprinted from "Elementary Seismology," C. F. Richter, 1958, with permission from W. H. Freeman and Company.

Figure C-3. The Richter Scale.

gists have introduced a new "physical" parameter called seismic moment, M_o, to describe the size of an earthquake. This parameter is related to the size of the fault rupture area, the average slip on the fault and the property in shear of the ruptured zone. Comparative values of the surface wave magnitudes and seismic moments of some famous earthquakes are given in table C-1.

Table C-1. Magnitude and seismic moment.

Earthquake	M_S	M_O
1960 Chili Earthquake	8.3 to 8.5	2.5×10^{30} dyne-cms
1964 Alaska Earthquake	8.3 to 8.4	7.5×10^{29} dyne-cms
1976 Tangshan Earthquake	7.8 to 8.0	1.0×10^{27} dyne-cms
1906 San Francisco Earthquake	8.2 to 8.3	1.0×10^{28} dyne-cms
1971 San Fernando Earthquake	6.4	1.0×10^{29} dyne-cms

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In order to relate this new size parameter with the existing magnitude scales, a moment magnitude (M_m) is introduced. In the M_L range of 5.5 to 7.0, M_m corresponds to M_L. M_m is related to seismic moment M_o by the following empirical relationship.

$$M_m = \tfrac{2}{3}\log M_o - 10.7 \qquad \text{(eq C-3)}$$

M_o is defined as:

$$M_o = GAS \qquad \text{(eq C-4)}$$

where

G = average shear modulus over the rupture zone

A = fault rupture area

S = average slip on the fault during the earthquake

e. Intensity measures. Another means of describing the size of an earthquake at a given location is the intensity scale. The two intensity scales used in the United States are:

—The Rossi-Forel Scale (RF Scale)
—The Modified Mercalli Scale (MM Scale)

Where the Modified Mercalli Scale is the most common. A simplified version of this scale is given in Table C-2. Table C-3 gives the Rossi-Forel scale. The russian scale is very similar to the MM scale. The RF scale which was developed in the late 19th century was used in this century until 1930. Since then, use of the MM scale has become more common. Table C-4 shows the approximate relationship between the MM scale and the RF scale. It is important to note that all of the above scales are subjectively assigned by investigators after observing and reviewing the earthquake effects in a given region. The assignment of proper intensity value therefore requires a careful analysis of the affected region. Unless the guidelines for assigning intensities are properly and correctly followed, there could be an error in the assigned value.

f. Relations for magnitude and intensity. Empirical relationships are available in the literature to relate the magnitude of an earthquake and the epicentral intensity. The following show such relationships.

Gutenberg and Richter (1956) (Biblio 87),

$$M_L = 1 + \tfrac{2}{3}I_o \qquad \text{(eq C-5)}$$

Krinitzky and Chang (1975) (Biblio 92),

$$M_L = 2.1 + \tfrac{1}{2}I_o \qquad \text{(eq C-6)}$$

Chinnery and Rogers (1973) for Northeastern United States (Biblio 85)

$$M_L = 1.2 + 0.6I_o \qquad \text{(eq C-7)}$$

where M_L = Richter Magnitude or local magnitude

I_o = Modified Mercalli Intensity in the epicentral area

All such relationships, including those derived for specific sites where specific data are avail-

Table C-2. The Modified Mercalli intensity scale.

Mercalli's (1902) improved intensity scale served as the basis for the scale advanced by Wood and Nuemann (1931), known as the modified Mercalli scale and commonly abbreviated MM. The modified version is described below with some improvements by Richter (1958).

To eliminate many verbal repetitions in the original scale, the following convention has been adopted. Each effect is named at the level of intensity at which it first appears frequently and characteristically. Each effect may be found less strongly or more often at the next higher grade. A few effects are named at two successive levels to indicate a more gradual increase.

Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick, or otherwise is specified by the following lettering (which has no connection with the conventional Class A, B, C construction).

Masonry A. Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.

Masonry B. Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.

Masonry C. Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

Masonry D. Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

Modified Mercalli Intensity Scale of 1931 (abriged and Rewritten by C. F. Richter).

I. Not felt. Marginal and long-period of large earthquakes.

II. Felt by persons at rest, on upper floors, or faborably placed.

III. Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.

IV. Hanging objects swing. Vibration like passing of heavy trucks or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of 4, wooden walls and frames crack.

V. Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.

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Table C-2. The Modified Mercalli intensity scale—continued.

VI. Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, and so on, off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, School). Trees, bushes shaken visibly or heard to rustle.

VII. Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices, unbraced parapets, and architectural ornaments. Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.

VIII. Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.

IX. General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. General damage to foundations. Frames racked. Conspicuous cracks in ground. In alluviated areas, sand and mud ejected, earthquake fountains, sand craters.

X. Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dame, dikes, embankments. Large landslides. Water thrown on bansk of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.

XI. Rails bent greatly. Underground pipelines completely out of service.

XII. Damage nearly total. Large rock masses displaced. Lines of sight.

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Table C-3. The Rossi-Forel scale.

The most commonly used form of the Rossi-Forel (R.F.) scale reads as follows:

I. Microsiesmic shock. Recorded by a single seismograph or by seismographs of the same model, but not by several seismographs of different kinds: the shock felt by an experienced observer.

II. Extremely feeble shock. Recorded by several seismographs of different kinds; felt by a small number of persons at rest.

III. Very feeble shock. Felt by several persons at rest; strong enough for the direction or duration to be appreciable.

IV. Feeble shock. Felt by persons in motion; disturbance of movable objects, doors, windows, cracking of ceilings.

V. Shock of moderate intensity. Felt generally by everyone; disturbance of furnature, beds, etc., ringing of some bells.

VI. Fairly strong shock. general awakening of those asleep; general ringing of bells; oscillation of chandeliers; stopping of clocks; visible agitation of trees and shrubs; some startled persons leaving their dwellings.

VII. Strong shock. Overthrow of movable objects; fall of plaster; ringing of church bells; general panic, without damage to buildings.

VIII. Very strong shock. Fall of chimneys; cracks in the walls and buildings.

IX. Extremely strong shock. Partial or total destruction of some buildings.

X. Shock of extreme intensity. Great disaster; ruins; disturbance of the strata, fissures in the ground, rock falls from mountains.

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Table C-4. The relation between Modified Mercalli intensity (MM) and Rossi-Forel intensity (RF).

MM	RF
I	I
II	I-II
III	III
IV	IV-V
V	V-VI
VI	VI-VII
VII	VIII
VIII	VIII+ to IX-
IX	IX+
X-XII	X

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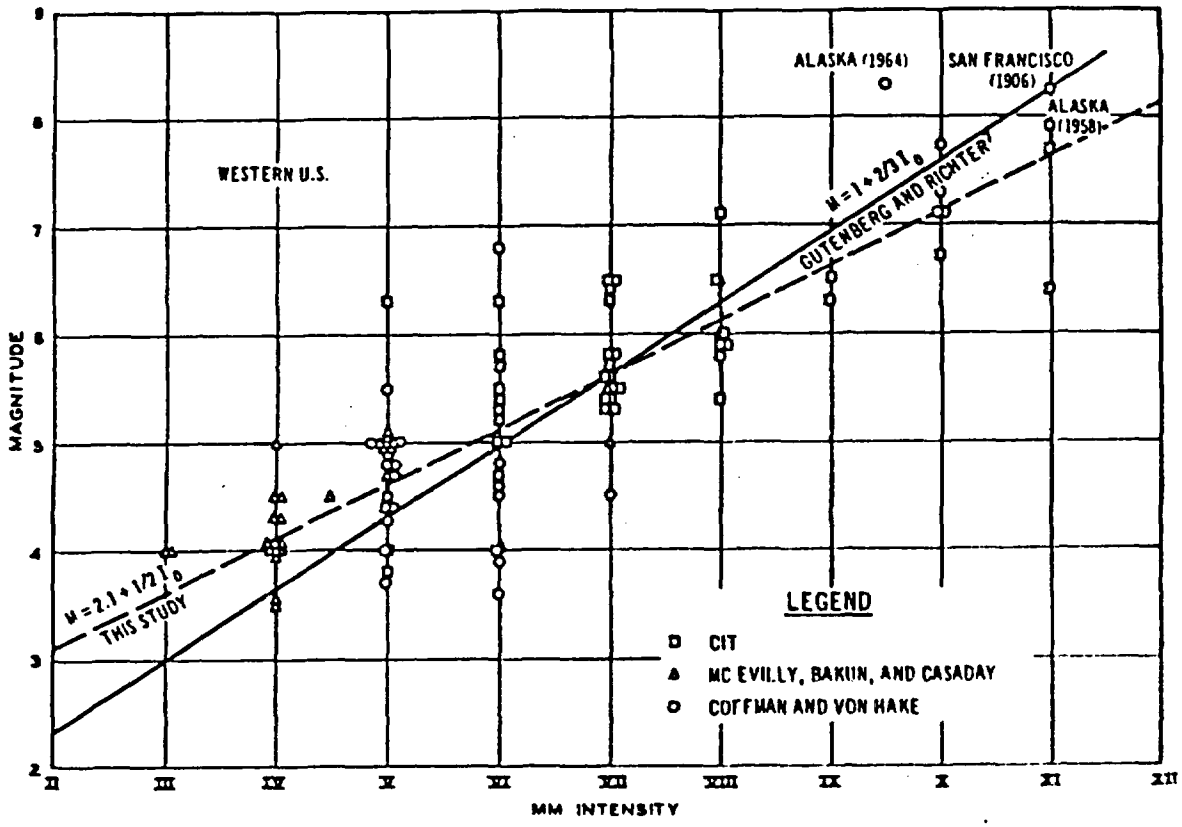
able, are extremely approximate and the scatter of data about the predicted lines is large. Note that much of the scatter is due to the necessity of empirically converting site intensity data to the equivalent I_o value at the epicentral area; so as to normalize the site distance attenuation effects. Figure C-4 (taken from Krinitzky and Chang, Biblio 91) shows the above relationships along with the data behavior.

g. Recording instruments for ground motion. With the introduction of modern strong motion instruments, the size of the ground motion at a given location is often expressed by means of the instrumentally recorded ground motion parameter. The most commonly used instruments for engineering purposes are the strong motion accelerographs. These instruments record the acceleration time history of ground motion at a site. Figure 2-1 of paragraph 2-3*b* shows a typical accelerogram recorded by such an instrument. By proper analysis of this acceleration time history to account for instrument bias and base line correction, the resulting corrected acceleration record can be used by engineers. This corrected acceleration record can yield ground velocity and ground displacement by appropriate integrations, see figures 2-1, and 2-2 in paragraph 2-3*b*.

h. Relations for recorded ground motion and intensity. To relate the instrumentally recorded parameters such as acceleration, velocity and displacement with intensity parameters, empirical equations have been developed by various researchers. It should be cautioned again that such relationships are obtained from widely scattered and sparse data and should only be used with recognition of their inherently large prediction error. From studies related to earthquake damage estimation and earthquake insurance, it has been observed that the Modified Mercalli intensity scale is the easiest and most convenient with which to work. Most of the



Reprinted from "Specifying Peak Motions for Design Earthquakes," Krinitzski, E. L. and Chang, F. K., Report No. 4 in the series, State-of-the-Art for Assessing Earthquake Hazards in the United States, U.S. Army Engineers Waterways Experiment Station, Misc. Paper S-73-1, 1975.

Figure C-4. Relation between earthquake magnitude and intensity.

available damage statistics are related to the MM intensity at a site. However, for the relatively recent instrumentally recorded data, the information on ground motion is usually in the form of a peak ground motion parameter such as the PGA, and many empirical relationships are available in the literature to relate the MM intensity with the PGA. Peak ground acceleration is an instrumentally recorded continuous variable whereas Modified Mercalli intensity is a subjectively assigned discrete integer variable. Thus, it should be expected that there will be a range or increment of continuous PGA values corresponding to a given intensity level. In the past, a number of researchers have developed PGA-MMI relationships. In each of the relationships given below, I is Modified Mercalli intensity and A is peak ground acceleration in cm/sec^2.

Gutenberg and Richter (1942) (Biblio 88)	$\log A = -0.5 + 0.33I$	(eq C-8)
Hershberger (1956) (Biblio 89)	$\log A = -0.9 + 0.43I$	(eq C-9)
Ambrasey (1974) (Biblio 84)	$\log A = -0.16 + 0.36I$	(eq C-10)
Trifunac and Brady (1975) (Biblio 103)	$\log A = 0.014 + 0.3I$	(eq C-11)

All the above relationships are log-linear in format. Recent work by McCann and Shah (Biblio 100) has shown that the assumption of a log-linear relationship between PGA and MMI may not be a reasonable one. Figure C-5 shows the following suggested relationship with two other relationships from above:

McCann and Shah (1979) (Biblio 100)	$\log A = -0.024I^2 + 0.595I - 0.68$	(eq C-12)

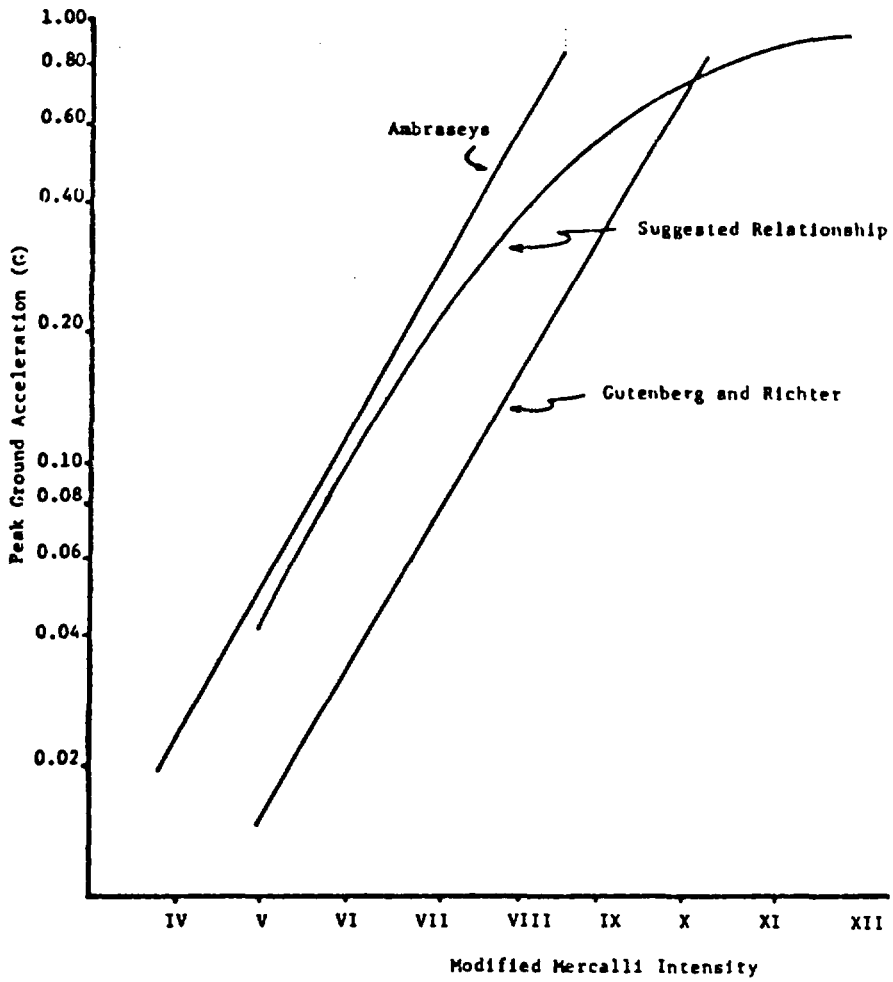


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Figure C-5. McCann and Shah relationship.

In this relationship, it is assumed that a range of peak ground acceleration values are associated with each intensity level. Figure C-6 shows the PGA-MMI relation and the interval associated with each intensity. Table C-5 lists this range of PGA values associated with each MMI level.

C-2. Response Spectrum Representation of Seismic Ground Motion at Site.

Seismic ground motion may be roughly characterized as a set of time-varying harmonic vibrations having a fairly broad range of frequencies. Structures subjected to this input motion tend to amplify the harmonics near their own natural frequencies and filter or attenuate the others. The resulting structural response therefore, depends upon the frequency content of the harmonics in the ground motion and their relation to the dynamic frequency characteristics of the structure. This paragraph provides the definitions and discussions of the response spectrum representation of this inter-relationship between ground motion input and structural response.

a. Single degree-of-freedom system response. Figure C-7 shows the system and the definition for seismic input and response.

(1) *Response to General Input x(t).* For any given ground acceleration $\ddot{x}(t)$, the relative displacement response is

$$u(t) = -\frac{1}{\omega_D}\int_0^t \ddot{x}(\tau)e^{-\beta\omega(t-\tau)} \sin[\tau_D(t-\tau)]d\tau \qquad \text{(eq C-13)}$$

and for the case of zero damping this equation simplifies to

$$u(t) = -\frac{1}{\omega}\int_0^t \ddot{x}(\tau)\sin[\omega(t-\tau)]d\tau \qquad \text{(eq C-14)}$$

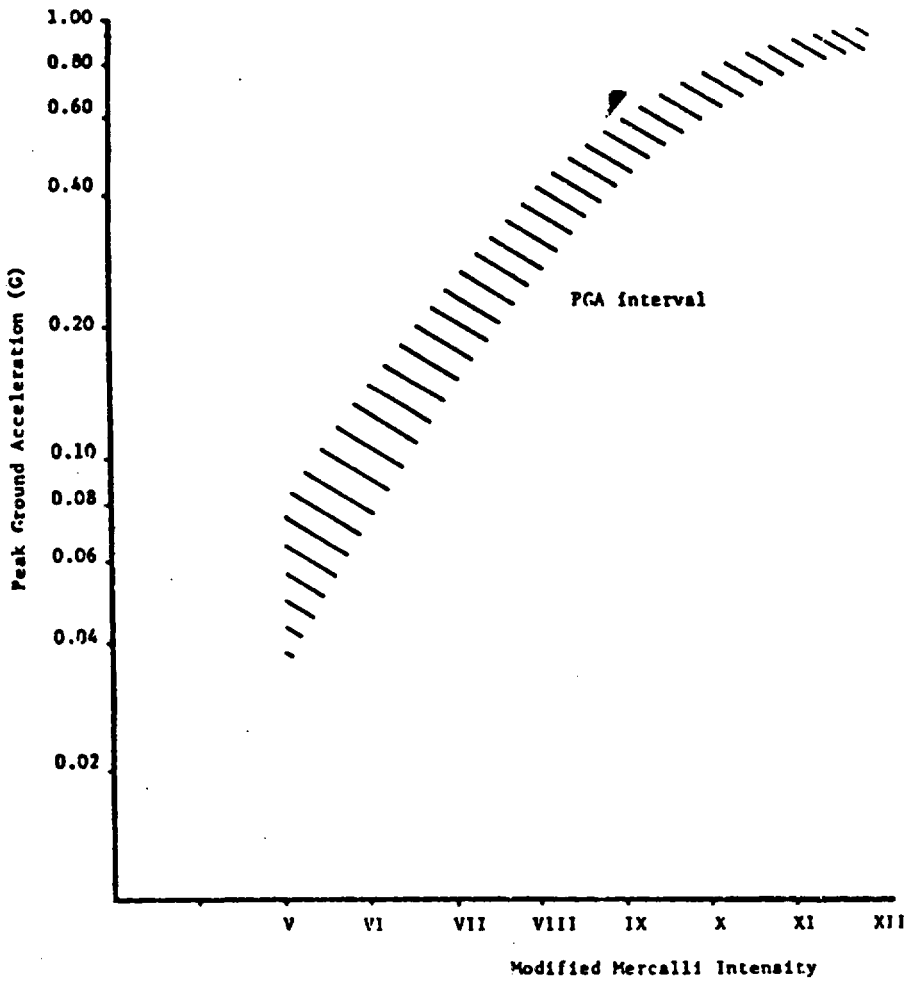

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Figure C-6. The PGA-MMI relationship shown with the intervals associated with each intensity.

Table C-5. Relationship between MMI and PGA.

MMI	PGA(in g unit)
V	$0.03 < A < 0.08$
VI	$0.08 < A < 0.15$
VII	$0.15 < A < 0.25$
VIII	$0.25 < A < 0.45$
IX	$0.45 < A < 0.60$
X	$0.60 < A < 0.80$
XI	$0.80 < A < 0.90$
XII	$A > 0.90$

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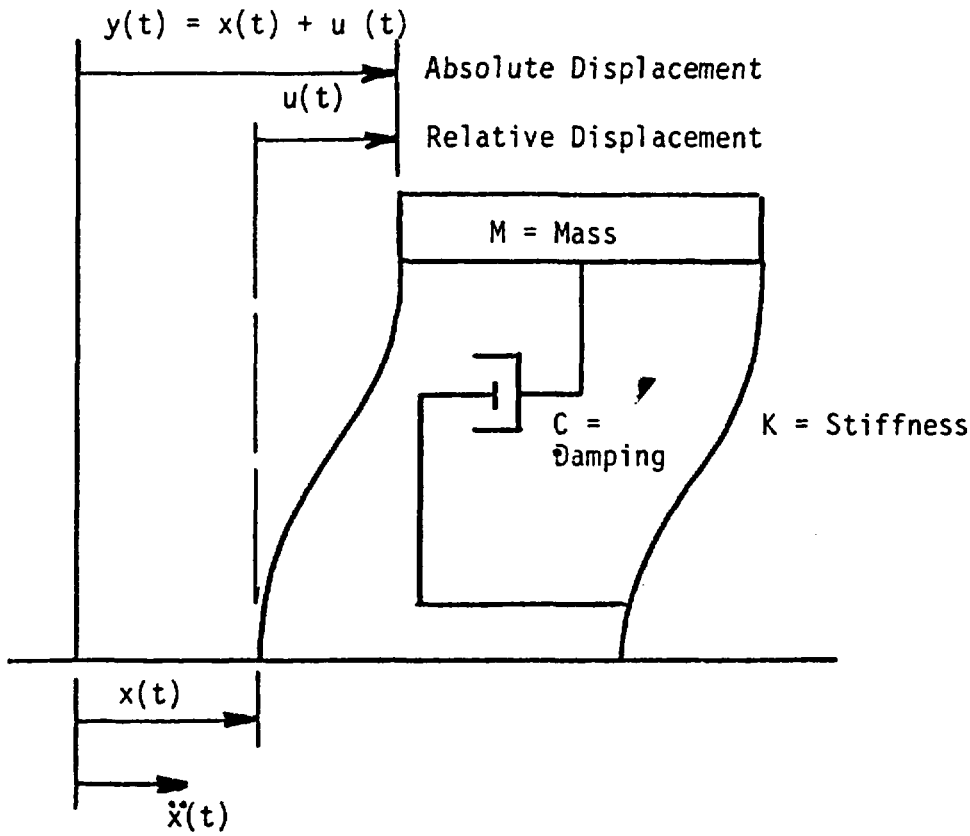

System Properties

$\omega = \sqrt{K/M}$ = undamped natural frequency

$\beta = \frac{C}{2M\omega}$ = fraction of critical damping

$\omega_D = \omega\sqrt{1-\beta^2}$ = damped natural frequency

Ground Motion

$x(t)$ = displacement

$\dot{x}(t) = \frac{dx}{dt}$ = velocity

$\ddot{x}(t) = \frac{d^2x}{dt^2}$ = acceleration

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Figure C-7. Single degree of freedom system.

Relative velocities and accelerations are given by the time derivatives $\dot{u}(t)$ and $\ddot{u}(t)$ respectively. ω_D is damped natural frequency.

(2) Response to Sinusoidal Input. If the ground acceleration $\ddot{x}(t)$ were to be a single unit amplitude sinusoid at frequency Ω

$\ddot{x}(t) = \sin\Omega t$ then the corresponding response is given by $u(t) = [H(\omega)]\sin[\Omega t + \phi]$

where ϕ is a phase angle and

$$H(\omega) = \frac{1}{[(1-\Omega^2/\omega^2)^2 + 4(\beta\Omega/\omega^2)^2]^{1/2}} \quad \text{(eq C-15)}$$

is the system frequency response function which either amplifies or attenuates the response according to the frequency Ω/ω ratio, and the damping ratio β, see figure C-8. This function is most useful in the explanation of how predominant harmonics in ground motion, due to special soil conditions, can amplify the ordinates of the response spectrum.

b. Response spectra. For a given ground acceleration $\ddot{x}(t)$ such as shown in figure 2-4, and given damping, the absolute maximum values found from the complete time history solution of equation C-13 provide the response spectrum values at the system frequency ω or period $T = \frac{2\pi}{\omega}$. A response spectrum is traditionally presented as a curve connecting the maximum response values for a continuous range of frequency or period values, such as shown in figures 2-4 of paragraph 2-3c.

The different response spectra are defined as:

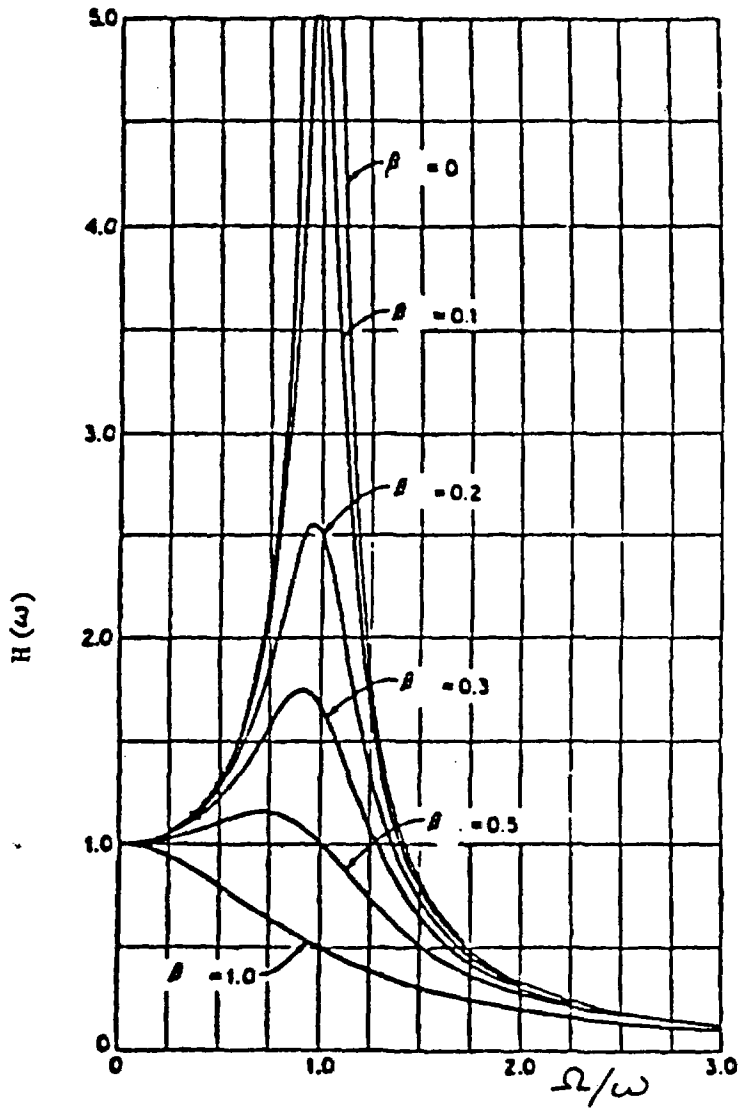

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Figure C-8. Maximum dynamic load factor for sinusoidal load.

$SD = u(t)_{max}$ = Relative Displacement Response Spectrum

$SV = \dot{u}(t)_{max}$ = Relative Velocity Response Spectrum

$SA = \ddot{y}(t)\ max = \ddot{u}(t) + \ddot{x}(t)\ max$ = Absolute Acceleration Response Spectrum

Then using the close approximation of $\omega = \omega_D$ for $\beta \leq 0.1$, the more commonly employed versions for engineering purposes are:

$S_V = \omega\ (SD)$ = Pseudovelocity Spectrum (eq C-16)

$S_a = \omega^2\ (SD)$ = Pseudovelocity Spectrum. (eq C-19)

For the common structural damping values, and the earthquake type of input motion, there is essential equality for the real and pseudo-values,

$S_V \cong SV$ (eq C-18)

$S_a \cong SA$ (eq C-19)

Of course, for long period structures, the velocity equality breaks down since S_V approaches zero, while SV approaches PGV. This is because relative displacement approaches the ground displacement value, and there is small motion of the mass. The relationships between SD, S_V, and S_a can be justified by the following physical behavior of the vibrating system. At maximum relative displacement SD, velocity is zero, and maximum spring force equals maximum intertia force,

$k(SD) = m\ S_a$

giving $S_V = k/m(SD) = \omega^2(SD)$ (eq C-20)

Detailed discussions on response spectra and their computation from accelerograms are given in (Biblios 7,3,12). An example of a typical accelerogram spectrum is shown in figure 2-4. Also because of the relations $S_a = \omega\ S_V = \omega^2 S_d$, it is possible to represent spectra on tri-partite log paper, see figure 3-29 in paragraph 3-6e(1).

C-3. Methods of forecasting earthquake ground motion.

The following methods of ground motion specification are employed by engineers for the seismic resistant design of structures ranging from nuclear facilities to ordinary buildings. Herein the term "ground motion" is used in its general sense to include both the time history and response spectrum representations of earthquake effects. Also, all methods require an initial specification of the acceptable risk of exceeding the structural performance levels such as the damage threshold, functionality level, and condemnation threshold, in order to establish the corresponding level of ground motion severity.

a. Selected representative ground motion. Given the structure site, its soil column conditions, and the geological description of the effective earthquake sources and their corresponding travel paths to the site: a set of time histories (commonly three to five) is selected so as to have reasonably similar soil columns, source and travel path characteristics, distances, and magnitudes with these conditions at the site. The magnitude is selected according to the performance and reliability criteria for the structure. Both actual records and artificially generated time histories are both used for the selected set.

(1) This method has the advantages of providing a definite set of structural response time histories or response spectra. These results may be averaged to provide a single description of forecasted structure performance. The set of response spectra may be averaged (arithmetically or graphically) to provide the most representative response spectrum ordinates in the particular period range of the structural system. This method does not require the use of attenuation equations and spectral (DAF) shapes with their high variances of prediction error.

(2) The disadvantages are that it is often difficult to find the representative records that would correspond to the particular site condition; and the end results are based on an average representation of a very small sample. Much depends upon how sincerely the engineer believes that the selected small sample can actually forecast the future ground motion. Further description and discussion is given in (Biblio 102).

b. Analytical site-soil column response. This method uses a somewhat similar method to that of the selected method in C-3*a*. The main difference is that the selected time histories must be representative of bed rock motion. For a given magnitude, a set of rock site accelerograms is selected (or scaled) so as to best represent the forecasted duration, amplitude and spectrum shape of the site bed rock motion. Then with the data from the site soil boring investigations, a dynamic model of the site soil column is formulated. This model is subjected to the set of bed rock motions and the resulting set of site surface time histories is obtained. These histories or their averaged (and smoothed) spectra are used for the structural input. The principal

advantage of this method is that it provides the best analytical representation of the effects of the site soil column on the surface response. The disadvantages are inherent in the selected specification of the limited set of bed rock time histories, and in the accuracy of the analytical model of the site soil column. The uncertainties due to a small data set to represent the future forecast are also present as in the method C-3*a*. (Biblios 93,98,99) give detailed discussions on this method. In the assignment of a particular weight, as will be discussed in paragraph C-3*f*, of preference for the spectral shape as provided by a site soil-column response analysis, the following items should be considered and assessed for validity and applicability:

(1) The time histories and scaling factor for bed-rock earthquake motion. Are the histories inclusive of duration and frequency content representative of the various possible sources and travel paths? Has the scaling factor (for PGA) been evaluated by a hazard analysis similar in quality to that used for surface ground motion?

(2) Soil-Column Model: Have adequate boring investigations and related tests been made to reasonably establish the dynamic model properties. Is there adequate geological information to supplement the boring data? Is the model appropriate for the site.

(3) Have a sufficient number of bed-rock time histories been used to establish a reasonably reliable statistical average and measure of dispersion of surface motion spectra.

c. Empirical forecasts from representative records. This method involves two basic steps: given the risk of exceedance, forecast a spectral scaling factor (PGA or EPA) corresponding to this risk; then apply this scaling factor to a response spectrum shape (DAF) representative of the general site soil column condition. The first step may be either "deterministic" such that the most severe magnitude event occurs on the source at the epicentral location nearest to the site: or may be probabilistic such that the union or combination of the probabilities of all the effective event magnitudes, sources, and epicentral locations is considered in the seismic hazard of the specified ground motion description (PGA) × (DAF). For a given magnitude of event M at a given source to site distance R, this method consists of:

(1) Attenuation of the spectral scaling parameter (such as PGA) to the site. These attenuation relations are derived from past data and vary according to the data used and the statistical model and fitting procedure (usually regression analysis). There is usually a large prediction error (50 to 100%) about the central or median predicted value.

(2) The PGA at the site is represenative of accelerogram peak records. This "instrumental" value is converted (by judgement) to an effective EPA value, which when used to scale the spectral (DAF) shape should produce a reliable structural response spectrum. With the "properly" formulated analytical model of the structure, this spectrum "should" provide a reliable estimate of the actual structural deformations that would result from the event or any one of the events included in the seismic hazard analysis (with stated risk of exceedance such as 10% in one-hundred years). This method is based on the statistical principle that the best prediction of the future is the average behavior of many past records. Despite the disadvantages listed below, it is a common practical way to forecasting and specifying ground motion. Its results may be modified by the results of the other methods given herein. The disadvantages are:

(a) The high prediction error in the attenuation equations for PGA.

(b) The high variability of the spectral shape DAF as obtained from the average of normalized spectra having roughly similar soil conditions. The method of normalizing the spectra to a common unity value of PGA contributes much to the high variability of the DAF shape.

d. Empirical forecasts of spectral ordinates. This method is a refinement of paragraph C-3*c*, where the response spectrum value S_a or S_v at a given period (rather than the zero period PGA value) is attenuated from source to site. The advantage is that the site spectrum is obtained directly in terms of: the source-to-site distance, the travel path geology, the event magnitude and the site soil conditions. It is not necessary to employ the highly variable empirical DAF spectral shape as needed by the method in C-3*c*. The disadvantage is that the attenuation relations for the spectrum ordinates are much more subject to prediction error than these relations for PGA. The available data for near-source spectra and corresponding spectra at various site distances is from only a few recent events (such as 1971 San Fernando and 1980 Imperial Valley). The data is therefore both sparse and very sensitive to the geological conditions of the region where the records were obtained.

e. Mathematical or theoretical modeling of the seismic event. This method models the source fracture size and sequence of rupture impulses. These impulses are then propagated by wave mechanics through a model of the source to site

path. This allows inclusion of all that is both theoretically and empirically known about source mechanics and site response (included are directivity and magnitude effects). Disadvantages are lack of data and knowledge concerning the faulting mechanism and the travel path geology.

f. Summary. For any actual site hazard study requiring specified ground motion description, the most popular methods are those in C–3*b* and C–3*c*. When both are used for a particular project, the individual results should be reviewed for consistency and resolution of significant differences. Of course any knowledge available from results of the other methods can contribute to this consistency and resolution process for the final ground motion specification. In actual practice, when there are two or more sources of spectral shapes, the smoothing and averaging process is done by judgement rather than by any formal statistical method, see figure C–9.

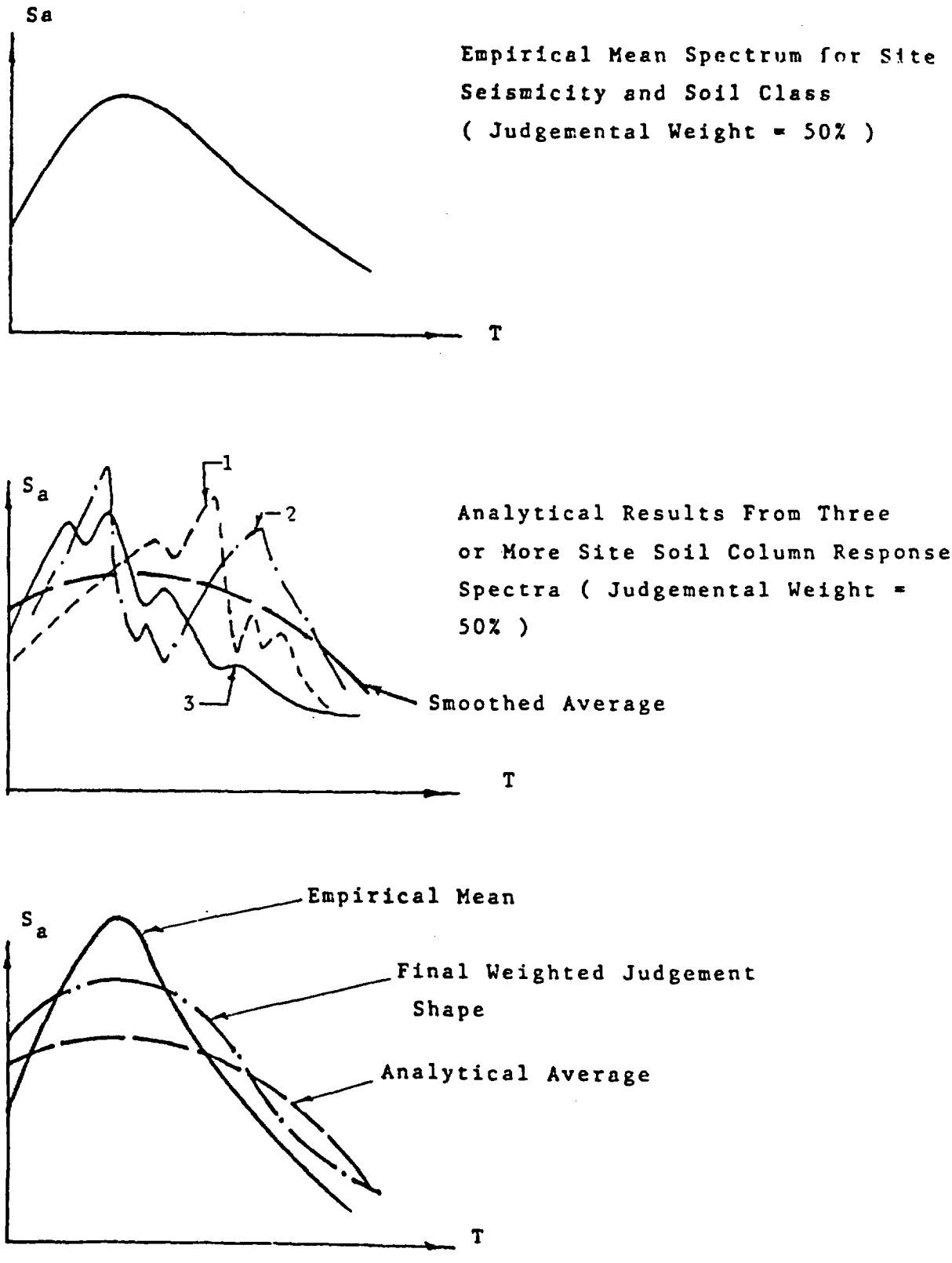

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Figure C-9. Judgemental averaging of empirical and analytical site spectra.

C-4. Emperical relations for seismicity and fault activity.

The following tables and figure are given to provide supplementary information concerning empirical relationships between fault length, fault displacement, and earthquake magnitude, Biblio (101) and degree of fault activity in terms of slip rate, Biblio (100).

Table C-6. Magnitude-displacement relation.

Equations of Best Straight-Line Fit for Magnitude Versus Log Displacement: M = a + b Log D

Fault	No.	a	b	Standard Deviation	Correlation Coefficient
North America	24	6.745	0.995	0.595	0.840
Rest of world	51	6.821	1.120	0.549	0.643
Worldwide	75	6.750	1.197	0.541	0.791
A normal-slip	20	6.827	1.050	0.449	0.777
B reverse-slip	11	7.002	0.986	0.469	0.744
C normal-oblique-slip	8	6.750	1.260	0.395	0.672
D reverse-oblique-slip	6	6.917	-0.150	0.421	-0.063
E strike-slip	30	6.717	1.214	0.639	0.814
A + C	28	6.757	1.226	0.431	0.774
B + D	17	6.846	1.023	0.506	0.674
C + D + E	44	6.705	1.206	0.586	0.794
C + D	14	6.692	1.165	0.451	0.568
B + E	41	6.767	1.200	0.606	0.811
A + C + E	58	6.737	1.221	0.549	0.806
B + D + E	47	6.742	1.188	0.597	0.795

Reprinted from "Fault and Earthquake Magnitude," Slemmons, D. B., State-of-the-Art for Assessing Earthquake Hazards in the United States, Report No. 6, Miscellaneous Paper S-73-1, U.S. Army Engineer Waterways Experiment Station, 1977.

Table C-7. Displacement-fault length relation.

Equations of Best Straight-Line Fit for Log Displacement Versus Log Length: Log D = a + b Log L

Fault	No.	a	b	Standard Deviation	Correlation Coefficient
North America	26	-4.720	1.036	0.632	0.737
Rest of world	48	-1.654	0.444	0.320	0.589
Worldwide	74	-3.185	0.747	0.515	0.645
A normal-slip	20	-4.375	1.014	0.567	0.620
B reverse-slip	9	-2.123	0.568	0.226	0.832
C normal-oblique-slip	8	-0.107	0.128	0.279	0.183
D reverse-oblique-slip	6	1.242	-0.220	0.154	-0.487
E strike-slip	31	-3.571	0.805	0.541	0.703
A + C	28	-2.898	0.705	0.351	0.685
B + D	15	-1.665	0.462	0.276	0.700
C + D + E	45	-2.924	0.684	0.516	0.624
C + D	14	0.033	0.081	0.265	0.130
B + E	40	-3.469	0.797	0.506	0.722
A + C + E	59	-3.239	0.756	0.474	0.680
B + D + E	46	-3.119	0.728	0.501	0.682

Reprinted from "Fault and Earthquake Magnitude," Slemmons, D. B., State-of-the-Art for Assessing Earthquake Hazards in the United States, Report No. 6, Miscellaneous Paper S-73-1, U.S. Army Engineer Waterways Experiment Station, 1977.

Table C-8. Magnitude-fault length relation.

Equations of Best Straight-Line Fit for Magnitude Versus Log Fault Length: $M = a + b \log L$

Fault	No.	a	b	Standard Deviation	Correlation Coefficient
North America	26	-0.146	1.504	0.628	0.815
Rest of world	49	2.971	0.920	0.500	0.680
Worldwide	75	1.606	1.182	0.603	0.724
A normal-slip	18	1.845	1.152	0.521	0.575
B reverse-slip	9	4.145	0.717	0.167	0.932
C normal-oblique-slip	10	3.117	0.913	0.457	0.604
D reverse-oblique-slip	7	4.398	0.568	0.340	0.522
E strike-slip	31	0.597	1.351	0.694	0.775
A + C	28	2.042	1.121	0.490	0.666
B + D	16	3.355	0.847	0.320	0.833
C + D + E	48	1.149	1.262	0.650	0.737
C + D	17	2.992	0.918	0.437	0.652
B + E	40	1.042	1.277	0.664	0.773
A + C + E	59	1.204	1.260	0.639	0.724
B + D + E	47	1.357	1.217	0.638	0.758

Reprinted from "Fault and Earthquake Magnitude," Slemmons, D. B., State-of-the-Art for Assessing Earthquake Hazards in the United States, Report No. 6, Miscellaneous Paper S-73-1, U.S. Army Engineer Waterways Experiment Station, 1977.

Table C-9. Magnitude-length times displacement relation.

Equations of Best Straight-Line Fit for Magnitude Versus Log Length Times Displacement: M = a + b Log LD

Fault	No.	a	b	Standard Deviation	Correlation Coefficient
North America	24	3.510	0.701	0.503	0.889
Rest of world	46	4.158	0.610	0.464	0.731
Worldwide	70	3.740	0.680	0.489	0.828
A normal-slip	18	4.551	0.530	0.421	0.750
B reverse-slip	9	5.310	0.423	0.213	0.886
C normal-oblique-slip	8	3.281	0.785	0.325	0.793
D reverse-oblique-slip	6	3.706	0.678	0.353	0.550
E strike-slip	29	3.220	0.759	0.567	0.859
A + C	26	3.691	0.707	0.388	0.792
B + D	15	4.478	0.550	0.327	0.834
C + D + E	43	3.238	0.766	0.510	0.850
C + D	14	3.168	0.802	0.340	0.784
B + E	38	3.424	0.728	0.536	0.859
A + C + E	55	3.393	0.745	0.503	0.837
B + D + E	44	3.441	0.726	0.515	0.853

Reprinted from "Fault and Earthquake Magnitude," Slemmons, D. B., State-of-the-Art for Assessing Earthquake Hazards in the United States, Report No. 6, Miscellaneous Paper S-73-1, U.S. Army Engineer Waterways Experiment Station, 1977.

Table C-10. Magnitude-length times squared displacement relation.

Equations of Best Straight-Line Fit for Magnitude Versus Log Length Times Square of Displacement: $M = a + b \log LD^2$

Fault	No.	a	b	Standard Deviation	Correlation Coefficient
North America	24	4.808	0.420	0.526	0.878
Rest of world	46	4.967	0.417	0.473	0.719
Worldwide	70	4.865	0.427	0.496	0.823
A normal-slip	18	5.568	0.299	0.427	0.742
B reverse-slip	9	5.865	0.289	0.242	0.850
C normal-oblique-slip	8	4.103	0.573	0.309	0.815
D reverse-oblique-slip	6	4.290	0.522	0.373	0.468
E strike-slip	29	4.491	0.480	0.574	0.855
A + C	26	4.752	0.459	0.384	0.796
B + D	15	5.162	0.382	0.350	0.808
C + D + E	43	4.473	0.489	0.513	0.848
C + D	14	3.985	0.590	0.340	0.785
B + E	38	4.597	0.468	0.535	0.859
A + C + E	55	4.582	0.477	0.499	0.840
B + D + E	44	4.587	0.469	0.516	0.852

Reprinted from "Fault and Earthquake Magnitude," Slemmons, D. B., State-of-the-Art for Assessing Earthquake Hazards in the United States, Report No. 6, Miscellaneous Paper S-73-1, U.S. Army Engineer Waterways Experiment Station, 1977.

Table C-11. Degree of fault activity.

FAULT	SLIP RATE (CM/YEAR)	CALCULATED CUMULATIVE SLIP (M)				MAX SLIP /EVENT (METERS)	RECURRENCE INTERVAL (YRS.)
		10K yrs	35K yrs	100K yrs	500K yrs		
Fairweather, Ak.	5.8	580	2030	5800	29000	10	170
San Andreas, Ca.	3.7	370	1295	3700	18500	10	270
Hayward, Ca.	.6	60	210	600	3000	2	300
Coyote Creek, Ca.	.3	30	105	300	1500	1.5	500
Lower Rhine Graben, Ger.	.023	2.3	8.5	23	115	.5	2000
Upper Rhine Graben, Ger.	.005	.5	1.75	5	25	.3	6000
Cleveland Hill, Ca.	.0006	.06	.21	.60	3	.24	30000
Rawhide Flat West, Ca.	.00025	.025	.087	.25	1.25	.08	32000
Negro Jack Point, Ca.	.00007	.007	.025	.07	.35	.02	29000

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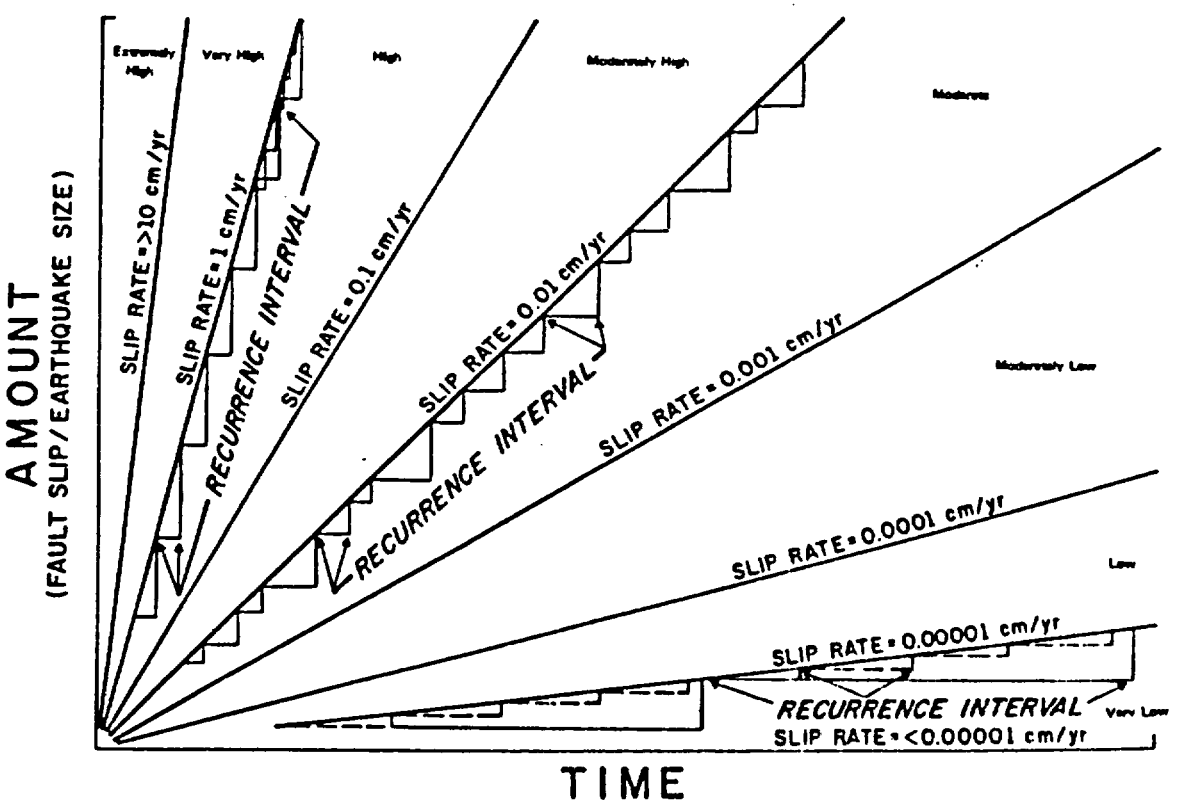

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Figure C-10. Relative degree of fault activity.

APPENDIX D
DESIGN EXAMPLES—GROUND MOTION

D-1. Purpose and Objectives.

The purpose of this appendix is to provide examples of the assumptions, procedures, and calculations required for each step of the probabilistic hazard analysis for site specific ground motion. Example 1 is a simplified version that shows hand calculations for all steps; it is intended to provide a direct understanding of how each successive value is obtained. Examples 2 and 3 represent the more detailed, actual types of hazard analyses necessitating the use of a computer program. Example 2 covers steps I and II and detail; and example 3 provides additional examples of steps I and II and then shows steps III and IV leading to the description of hazard as the complementary cumulative distribution function or hazard curve for site PGA.

D-2. Introduction for Simplified Example 1.

The purpose of this example is to show a simple, by-hand set of calculations for each of the steps I through IV for a site hazard analysis. A point is to be determined on the hazard curve (fig 3-39), for P [PGA < PGA_j] with PGA_j = 0.20g, for an exposure time of t = 50 years. Then assuming that the complete hazard curve has been determined from a set of similarly calculated values of PGA_j, a selected response spectrum shape is scaled to illustrate step V, and provide an EQ-I site specific spectrum.

a. Step I. Identification and Modeling of Seismic Sources (para 3-4*b*). The building site is located in a region containing two distinct sources of seismicity; a line source 1, and an area source 2. Source 1 has been identified by the surface trace and subsurface geological structure of a strike-slip fault along with a history of earthquake reports and records associated with this fault. Source 2 is a general area within which a history of earthquake reports have occured; there may be faults with this area, however there is no surface evidence of their location. Figure D-1 shows the line and area models of sources 1 and 2, the estimated epicentral locations of past earthquakes along with the listings of historical records of earthquakes assigned to each source.

b. Step II. Evaluation of source seismicity and recurrence relations (para 3-4c). As shown in figure D-1, the line source 1 has a period of t_1 = 150 years of reported seismic events and records along its assigned length L_1 = 30 kilometers. The older reports in terms of intensity have been converted to equivalent magnitude values M, and the more recent events have directly measured magnitudes. Based on the fault length, along with its depth and slip activity, a maximum magnitude of M = 7.5 is assigned for this source. Area source 2 has a period of t_2 = 300 years of reported history. All events except the last one are in terms of MMI intensity I_o, and the last event has a measured magnitude. The MMI values are converted to equivalent local magnitude values by use of the Gutenberg-Richter equation C-5 given in appendix C. The geological structure within source 2 is judged to be capable of a maximum magnitude of M = 6.5. The recurrence relation for source 1 is developed by linear regression analysis as follows. The eight recorded events are ranked according to descending magnitude values such that the number N of events having magnitudes equal to greater than a given ranked magnitude is the ranked order number. These data are shown in figure D-2 along with the corresponding logarithm values ln N. A plot of ln N versus M in figure D-3 shows that a single straight line can represent the source 1. recurrence relation

$$\ln N_1 = \alpha_1 + \beta_1 M$$

Letting y = ln N_1 and x = M, the linear regression analysis calculations for the least-squared-error line

$$y = \alpha_1 + \beta_1 \chi$$

are shown in figure D-2, along with the normalization required to give

$$\ln N_i = \alpha_i + \beta_1 M = 1.29 - 1.32M$$

for a one kilometer, one year basis. A similar processing of the source 2 data provided the recurrence relation

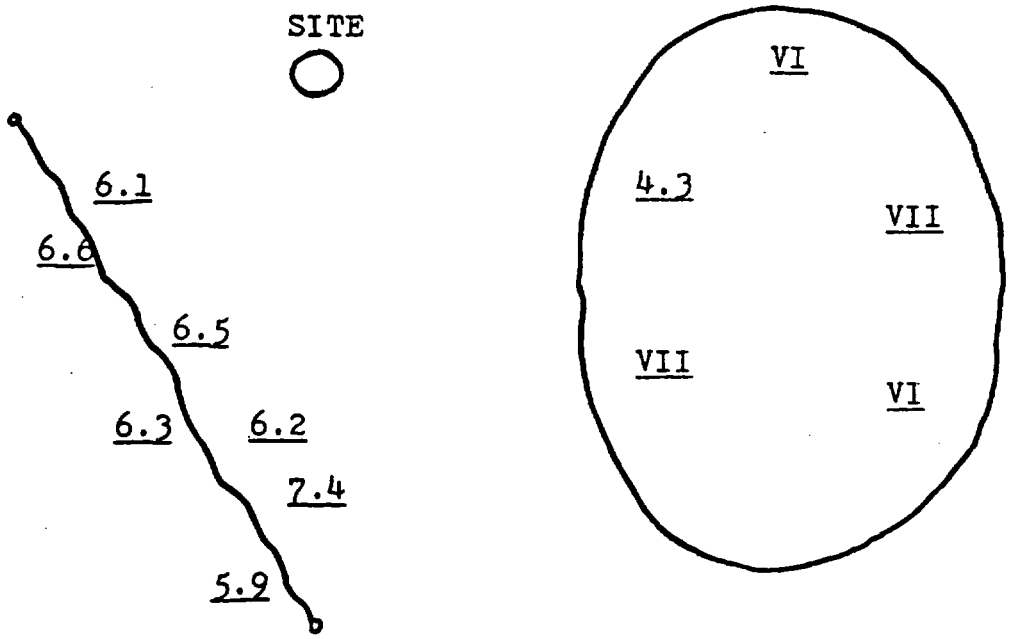
$$\ln N_2 = 5.81 - 0.95M$$

for the 300 year time period and the 400 square kilometer area. Normalization then gave

$$\ln N'_2 = \alpha'_2 + \beta_2 M = -5.89 - 0.95M$$

for a one square kilometer, one year basis.

c. Step III. Probabilistic Forecasting Model (para 3-4d). The Poisson occurrence model is assumed to forecast the probabilities of magnitude levels for both sources 1 and 2. Referring to equation 3-14 of paragraph 3-4d; given a length increment ΔL and the future time period t for source 1, the probability of no events greater



LINE SOURCE 1.
Length=30 km
150 Years of Record

Date	M
1830	6.5
1852	6.1
1871	6.6
1890	6.2
1911	5.9
1920	6.3
1946	7.4
1980	5.7

AREA SOURCE 2.
Area=400 sq km
300 Years of Record

Date	I_o	M*
1682	VII	5.7
1765	VI	5.0
1812	VI	5.0
1920	VII	5.7
1982	V	4.3

* $M=1.0+(2/3)I_o$, (eq C-5)

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Figure D-1. Source models and records for sources 1 and 2.

LINE SOURCE 1.

$L_1 = 30$ km, $T_1 = 150$ years, $M_{max} = 7.5$

N_1	$\ln N_1 = y$	$M = x$	$(y-\bar{y})$	$(x-\bar{x})$	$(x-\bar{x})^2$	$(y-\bar{y})(x-\bar{x})$
1	0	7.4	-1.33	1.06	1.12	-1.41
2	0.69	6.6	-0.64	0.26	0.07	-0.15
3	1.10	6.5	-0.23	0.16	0.03	-0.04
4	1.39	6.3	0.06	-0.04	0	0
5	1.61	6.2	0.28	-0.14	0.02	-0.04
6	1.79	6.1	0.46	-0.24	0.06	-0.11
7	1.95	5.9	0.62	-0.44	0.19	-0.27
8	2.08	5.7	0.75	-0.64	0.41	-0.48
$\Sigma =$	10.61	50.7		$\Sigma =$	1.90	-2.50

$$\bar{y} = \frac{10.61}{8} = 1.33, \quad \bar{x} = \frac{50.7}{8} = 6.34$$

$$\beta_1 = \frac{\Sigma(y-\bar{y})(x-\bar{x})}{\Sigma(x-\bar{x})^2} = \frac{-2.50}{1.90} = -1.32$$

$$\alpha_1 = \bar{y} - \beta_1 \bar{x} = 1.33 - (-1.32)(6.34) = 9.70$$

$$\ln N_1 = \alpha_1 + \beta_1 M = 9.70 - 1.32 M$$

Normalizing to a 1 km, 1 year Basis, using (eq 3-5);

$$\alpha_1' = \alpha_1 - \ln(L_1 T_1) = 9.70 - \ln(4500) = 1.29$$

$$\ln N_1'(m) = \alpha_1' + \beta_1 m = 1.29 - 1.32 m$$

for $m \leq 7.5$

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Figure D-2. Recurrence relation calculations for source 1.

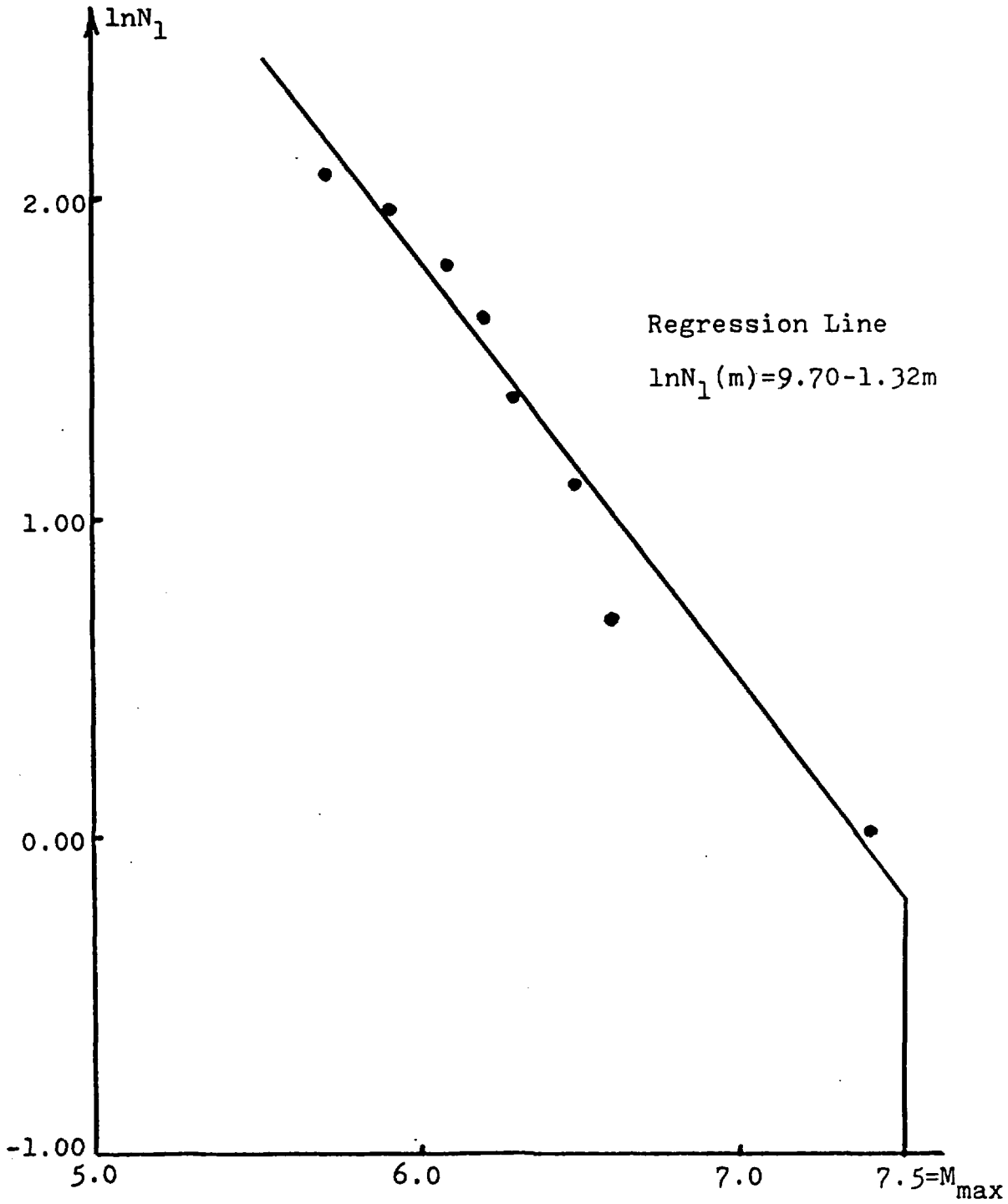

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Figure D-3. Recurrence data plot for source 2.

than $M_1 = m$ is

$$P[M_1 \leq m] = P(o,m,t) = \exp[-N'_1(m)\,\Delta L\,t]$$

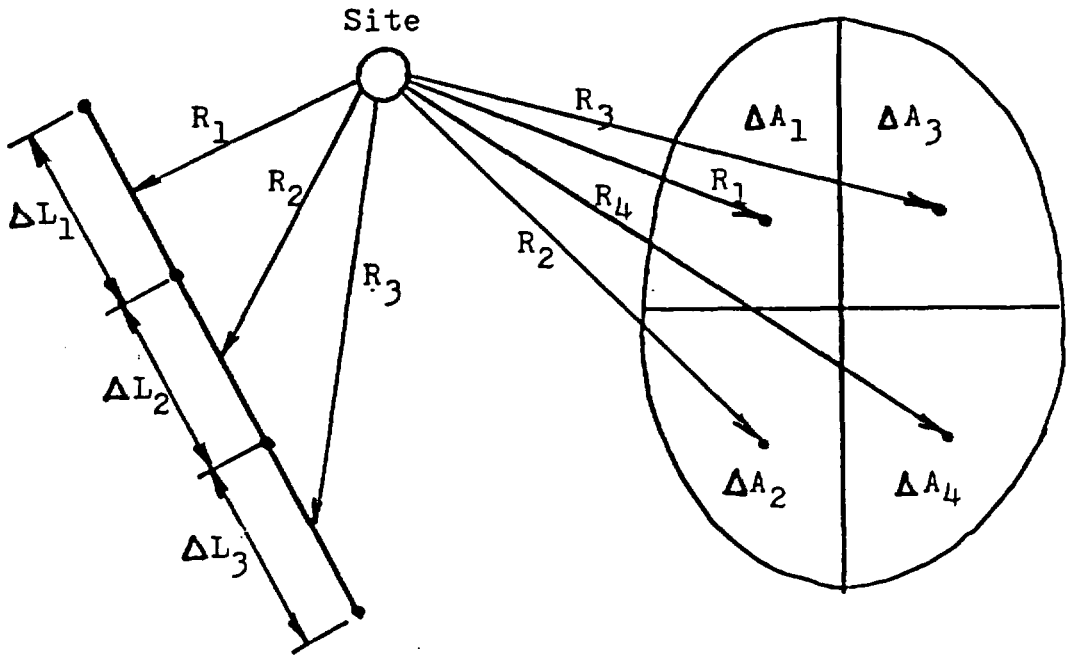
where $N'_1(m) = \alpha'_1 + \beta_1 m$.

Similarly given an area increment ΔA and t for source 2,

$$P[M_2 \leq m] = P(o,m,t) = \exp[-N'_2(m)\,\Delta A\,t]$$

where $N'_2(m) = \alpha'_2 + \beta_2 m$

The value of magnitude m to be employed in these equations is that which can produce the attenuated value of PGA = 0.20g at the building site when the earthquake event occurs at the center of the increments ΔL and ΔA of sources 1 and 2 respectively. In order to determine these magnitudes it is necessary to divide the sources into elements, measure the element-to-site distance R, and then use the attenuation relation in Step IV. Figure D-4 shows the element modeling of the sources.



LINE SOURCE 1.
L=30 km
n=3 Elements
ΔL=10 km
Transmission Path A

AREA SOURCE 2.
A=400 sq km
n=4 Elements
ΔA=100 sq km
Transmission Path B

For the given PGA_j=0.20g ,the OASES attenuation curves in figure 3-23 provide the magnitudes m_i for each of the measured element to site distances R_i.

SOURCE 1.

i	R_i km	m_i
1	15	6.5
2	18	6.7
3	24	7.2

SOURCE 2.

i	R_i km	m_i
1	22	5.0
2	28	5.3
3	32	5.7
4	37	6.1

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Figure D-4. Source location and element properties.

d. Step IV Selection of the Attenuation Relation (para 3-5). The OASES relationship given by equation 3-21 and as shown in figure 3-23 has been judged to be appropriate for the source depth, travel path, and site soil characteristics. With the measured source element-to-site distances R given in figure D-4, and the given objective PGA = 0.20g = 196cm per second squared, the corresponding magnitude values can be found by interpolation between the curves of figure 3-23. The results are tabulated in figure D-4.

e. Combination of element and source probabilities. With the magnitudes m necessary to produce PGA = 0.20g at the site, the normalized recurrence relations are used to evaluate the corresponding rate values of $N'_1(m)$ and $N'_2(m)$ for use in the Poisson probability equations; these rates are tabulated in figure D-5.

The total hazard P [PGA > 0.20g] is calculated by 1 − P [PGA ≤ 0.20g], where P [PGA ≤ 0.20g] is the total probability of no exceedence of 0.20g at the site. This total probability is the probability of the intersection or mutual occurence of the occurences of ($M \leq m_i$) at all of the elements ΔL_i and ΔA_i of sources 1 and 2 respectively. In order to evaluate this intersection probability, an independent point source model is assumed for elements ΔL_i and ΔA_i. Accordingly, for the given level of PGA = 0.20g and the future time t = 50 years, the elements ΔL_i and ΔA_i are considered as point sources with seismicity rate $N'_1(m_i)\,\Delta L\,t$ and $N'_2(m_i)\,\Delta A\,t$ respectively. Here for each element the m_i is the magnitude level necessary to produce 0.20g at the site. Having the normalized rates $N'_1(m_i)$ and $N'_2(m_i)$ from the recurrence relations, the individual element probabilities of no magnitudes m_i capable of exceeding 0.20g at the site are:

$$P\,[M_1 \leq m_i] = \exp\,[-N'_1\,(m_i\,\Delta L\,t]$$

for elements ΔL_i on source 1 and

$$P\,[M_2 \leq m_i] = \exp\,[-N'_2\,(m_1)\,\Delta A\,t$$

for elements ΔA_i on source 2. Since each point source is assumed to be independent of the occurences of events on the other point sources, the intersection probability P [PGA ≤ 0.20g] for each source 1 and 2 is found by the product of the individual element probabilities for each source: P [PGA ≤ 0.20g] due to source 1 is the product of all of the (i = 1, 2, 3) element probabilities $\exp\,[-N'_1\,(m_i)\,\Delta L\,t]$ and equals (because exponents are added), $\exp\,[-\Sigma N'_1\,(m_i)\,\Delta L\,t]$. Similarly P[PGA ≤ 0.20g] due to source 2 is $\exp\,[-\Sigma N'_2\,(m_i)\,\Delta A\,t]$. Finally since each source is independent of events that may occur on the other source, the total probability at the site is

$$P[PGA \leq 0.20g] = \underset{\text{Source 1}}{P[PGA \leq 0.20g]}\ \underset{\text{Source 2}}{P[PGA \leq 0.20g]}$$

and hazard P[PGA>0.20g] is 1 − P [PGA≤0.20g].

The complete set of calculations is shown in figure D-5.

f. Construction of the site hazard curve. The calculations as performed for PGA_j = 0.20g, are repeated to evaluate P [PGA > PGA_j] for successive incremented values of PGA_j such as (0.10g, 0.15g, 0.25g, and 0.30g). The site hazard curve is drawn through the plot of the calculated hazard values verses their respective PGA_j values, as shown in figure D-6.

Since this curve is for the exposure time of t = 50 years which corresponds to the exposure time for EQ-I, the spectral scaling value PGA_I for this level of ground motion can be taken directly from the curve at the 50 percent hazard value. The curve gives PGA_I = 0.23g.

g. Step V Site Specific Response Spectrum for EQ-I. The soil conditions correspond to those for the soil class 1 as defined in paragraph 3-6*f*(3). It is therefore judged that the Kiremidjian and Shah mean DAF shape in figure 3-35, for the soil class = 1, damping = 5% is appropriate for the site. Having the scaling PGA_I = 0.23g, the EQ-I acceleration response spectrum S_{aI} is found by multiplying the selected DAF shape by 0.23g. This S_{aI} is shown in figure D-6. It should also be mentioned that the ATC 3-06 response spectrum shape (para 3-8) for the soil type S_2, as scaled by the PGA_I = 0.23g, would have been suitable for this site.

D-3. Introduction for Computer Examples 2 and 3.

It is assumed that computer programs for seismic hazard analysis such as the Stanford Seismic Hazard Analysis = STASHA, are available for use. A complete flow chart describing the seismic hazard methodology is presented. This will be followed by numerical examples describing the separate stages of the model. It is important to note that computer programs must be available to conduct the probabilistic hazard analysis as outlined in paragraphs 3-3 through 3-5. Figure D-7 shows the general flow chart for seismic hazard analysis. Figure D-8 shows further subtasks within each of the three stages outlined in figure D-7. In most of the available computer programs, the plotting programs are usually system dependent. In the examples, it will be assumed that stage I, the raw data, has

SOURCE 1. $\Delta L = 10$ km, $t = 50$ years

$$\ln N_1'(m) = 1.29 - 1.32\,m$$

Elem. i	m_i	$\ln N_1'(m)$	$N_1'(m) \times 10^4$	$N_1'(m) \cdot \Delta L \cdot t$
1	6.5	−7.29	6.82	0.341
2	6.7	−7.55	5.26	0.263
3	7.2	−8.21	2.72	0.136

$$\Sigma N_1'(m) \cdot \Delta L \cdot t = 0.740$$

$P[PGA \le 0.20g]$ due to source 1.

$$= \exp\left[-\Sigma N_1'(m) \cdot \Delta L \cdot t\right] = \exp[-0.740] = \underline{\underline{0.477}}$$

SOURCE 2. $\Delta A = 100$ sq km, $t = 50$ years

$$\ln N_2'(m) = -5.89 - 0.95\,m$$

Elem i	m_i	$\ln N_2'(m)$	$N_2'(m) \times 10^5$	$N_2'(m) \cdot \Delta A \cdot t$
1	5.0	−10.64	2.39	0.120
2	5.3	−10.93	1.79	0.090
3	5.7	−11.31	1.22	0.061
4	6.1	−11.68	0.85	0.043

$$\Sigma N_2'(m) \cdot \Delta A \cdot t = 0.314$$

$P[PGA \le 0.20g]$ due to source 2.

$$= \exp\left[-\Sigma N_2'(m) \cdot \Delta A \cdot t\right] = \exp[-0.314] = \underline{\underline{0.731}}$$

$P[PGA \le 0.20g]$ due to both sources

$$= (0.477)(0.731) = 0.349$$

$$\text{Hazard} = P[PGA > 0.20g] = 1 - 0.349 = \underline{\underline{0.651}}$$

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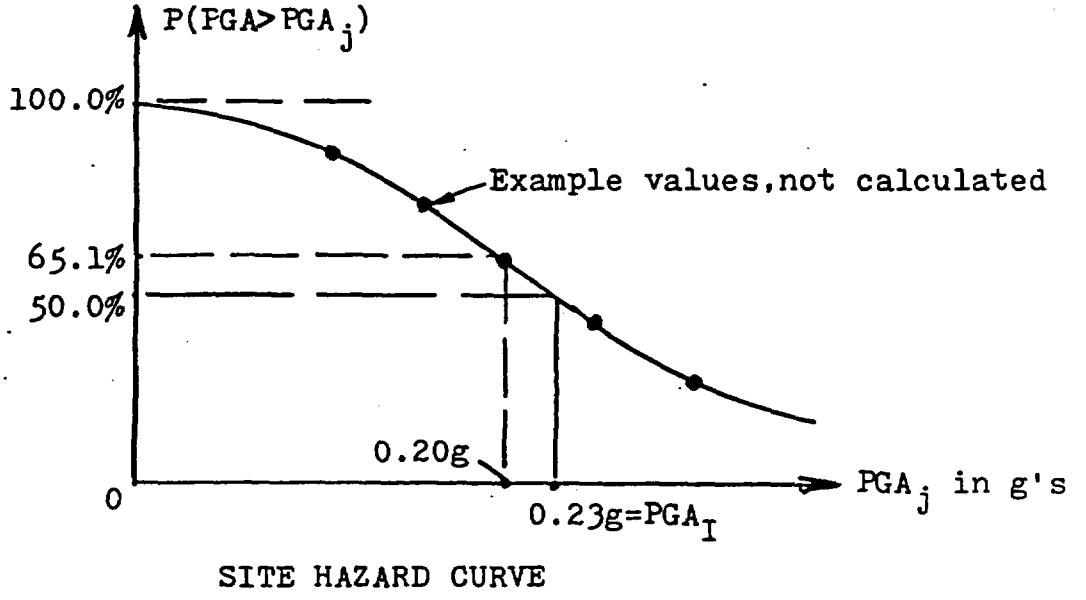
Figure D-5. Probability calculations for event combinations giving the hazard P [PGA > 0.20g].



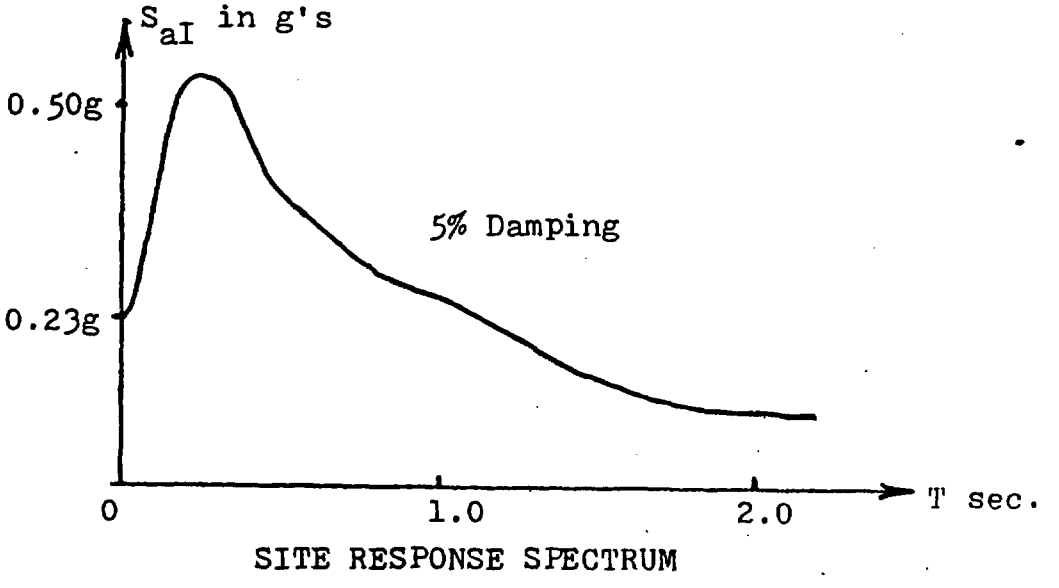


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Figure D-6. Site hazard curve and scaled site spectrum for EQ-I.

already been treated and that the seismic sources of stage II have been identified; these correspond to step I of the hazard analysis. The next section will give an example of how one determines the recurrence relationship for the identified seismic sources; step II of the hazard analysis.

D-4. Example 2.

Figure D-9 shows a listing of earthquakes for a region between 1850 and 1967. There were 18 events with magnitudes between 3 and 5.5. The data base is for a 125 year time period. The format in which the data is read is given in section 6.3 of STASHA. A log-linear recurrence relationship of the form needs to be fitted to these data (Step II). The analyst does not wish to normalize with respect to the source length (or area) or the time period over which the data was available; (See para 3-4 for normalization). A magnitude increment of 0.2 is used to compute the cumulative histogram. It is assumed in this example that a single log-linear line will suffice to describe the source seismicity. An upper cutoff magnitude of 6.5 (which is obtained from geological considerations) is given for the source; (see para 3-3). Figure D-10 shows the output of the computer program which gives the recurrence relationship. The following nomenclature is used in figure D-10.

NBRC = Number of earthquake records used in the analysis.

AREA = Area or length of the seismic source under consideration. (In this example, it is shown as zero since normalization of α is not needed)

RMBK = Breakoff magnitude

X-Mean = Mean of the independent variable (Richter magnitude in this case)

Y-Mean = Mean of the dependent variable (number of earthquakes, log-scale)

XVAR = Variance of independent variable.

YVAR = Variance of dependent variable.

COVARXY = Covariance for X and Y.

VAR(LNNM) = Variance of the log to the base e of the cumulative number of occurrences.

STDV(LNNM) = Standard deviation of (LNNM).

CONF. VALUE = Value of t-student's distribution for the fitted line.

UPCNF = Value of upper confidence interval for a given RM.

DNCNF = Value of lower confidence interval for a given RM.

Figure D-11 shows the fitted recurrence line together with the data points and the confidence interval. Note that the regression line is extended beyond the last data point in order to intercept the cutoff magnitude line. In the above example, the RMBK, the breakpoint for the Richter magnitude was defined as zero; (See fig D-10). This indicates that only one single line was used to relate ln N(m) to m. Close examination of figure D-11 shows that the regression line does not fit well to the data. For example, for the magnitude range between 4 and 5, the fitted line underestimates the cumulative number of occurrences, and beyound the 5.0 magnitude the fitted line overestimates the cumulative number of occurrences. Thus, it seems reasonable to try a bi-linear fit with RMBK at 4.2. Figure D-12 shows the new output format and figure D-13 shows the bilinear fit. The resulting recurrence lines provide the mean number or rate of events equal to or above Richter magnitude m. This rate is used in the Poisson model (para 3-4) to estimate the probability of future activity for a given source (Step III).

D-5. Example 3.

In this example, the seismic hazard at a site in terms of probabilistic peak ground acceleration will be obtained. Figure D-14 shows a seismic region with two line sources and one area source. Occurrence data for each of the sources are given in figure D-15. The seismic sources were modelled after correlating past events to major fault systems and the tectonic features identified within the region (Step I). The future seismic exposure (PGA) for "CITY2" (see fig D-14) for a time period of 50 years is required. For this purpose, the following assumptions are made:

a. Past earthquake events (as recorded for the region) have been classified as shallow with hypocenters between 0 and 15km.

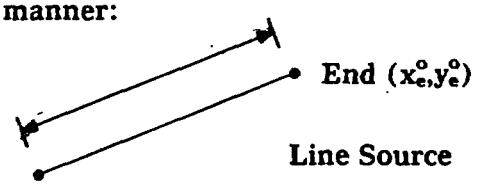
b. The average depth of the three seismic sources has been set equal to 10 km (0.087 degrees for the particular geographic location).

c. The length in degrees of the two line sources are, respectively:

Line Source 1 = 0.871°

Line Source 2 = 0.764°

These lengths have been obtained in the following manner:



$$L = \sqrt{(x_e^o - x_o^o)^2 + (y_e^o - y_o^o)^2}$$

d. The radius (in degrees) of the area source is

$$R = 0.749°$$

and is defined as the distance from the centroid of the epicenters associated to the source to the most distant epicenter in the source.

e. From regression analysis the following recurrence coefficients have been obtained (Step II).

Line Source 1 (bi-linear recurrence relationship)

ALPHA1 = 2.58, BETA1 = -1.09, ALPHA2 = 24.00, BETA = -4.55

Cutoff magnitude = 6.8, breakpoint magnitude = 6.45

Line Source 2 (bi-linear recurrence relationship)

ALPHA1 = 3.17, BETA1 = -0.74, ALPHA2 = 79.15, BETA2 = -12.4

Cutoff magnitude = 7.8, breakpoint magnitude = 6.50

Area Source 1 (bi-linear recurrence relationship)

ALPHA1 = 0.14, BETA1 = -0.07, ALPHA2 = 79.90, BETA2 = -13.04

Cutoff magnitude = 6.5, breakpoint magnitude = 6.15

All alpha values have been normalized with respect to time t = 50 years and the length (in degrees) or area of source, and the resulting recurrence rates are used in the Poisson probability model (Step III).

f. The attenuation parameters b_1, b_2, b_3, and c in eq. 3-21 for PGA are as follows (Step IV):

b_1 = 0.00429937

b_2 = 0.800

b_3 = 2.000

c = 0.3673769

g. Coordinates for sources and site.

Line Source 1:	X-coordinate of origin = 30.50° (longitude)
	Y-coordinate of origin = 31.97° (latitude)
	X-coordinate or end = 30.92° (longitude)
	Y-coordinate or end = 32.62° (latitude)
Line Source 2:	X-coordinate of origin = 30.51° (longitude)
	Y-coordinate of origin = 31.75° (latitude)
	X-coordinate of end = 31.30° (longitude)
	Y-coordinate of end = 31.00° (latitude)
Area Source 1:	X-coordinate of center = 32.39° (longitude)
	Y-coordinate of center = 31.078° (latitude)
Site (City2):	X-coordinate = 32.00° (longitude)

h. The input data format is given in section 7-2 of STASHA. Figure D-16 shows the listing of the output program ACC.LINE.AREA (STASHA, 1979). The output contains the input parameters plus the probabilities of exceedance and non-exceedance for each discrete value of the ground parameter of interest (PGA discretized at 0.05g intervals) under the heading "Probability Distribution of Peak Ground Acceleration". Figure D-17 shows a plot of the complementary cumulative distribution function or hazard curve for the City 2. From figure D-16,

$$P(A < 0.10g) = 0.7512$$

Thus, for city 2, there is an approximately 75% chance of exceeding 0.10g at least once during the next 50 years, or 25% chance of not exceeding 0.10g during the same time period. Hence,

P(zero exceedance of 0.10g in 50 years) = 0.25

(1) From the binomial probability law, it is known that for independent trials with probability of success p at each trail, the probability of r successes in n trials is given by

$$P_n(r) = \binom{n}{r} p^r (1-p)^{n-r}$$

where r = 0, 1, 2, . . ., n and $\binom{n}{r} = \dfrac{n!}{r!\,(n-r)!}$

(2) Let each trial be a one-year duration for which we are observing the level of peak ground acceleration. Define success as that event when the peak ground acceleration for a given trial

(year) exceeds 0.10g. Thus, the probability of zero successes in 50 years is the same as the probability of zero successes in 50 trials. Hence,

$$P_{50}(o) = \binom{50}{o} p^{o} (1-p)^{50} = (1-p)^{50}$$

Then having

$$P_{50} = 0.25 = (1-p)^{50}$$

giving

$$p = 0.027$$

Therefore, for CITY2, there is a 2.7 percent chance that in any given year, a peak ground acceleration of 0.10g will be exceeded. The corresponding Return Period RP in "CITY2" for a peak ground acceleration of 0.10g is

$$1/0.027 = 37 \text{ years}$$

(3) Similarly, using the complementary cumulative distribution function computed for "CITY2", a table of peak ground acceleration and return period can be developed and plotted to obtain a curve referred to as an Acceleration Z one Graph (AZG). Table D-1 and figure D-18 show the values of Return Period versus PGA and the AZG for "CITY2." Using this figure D-18, the PGA_I for EQ-I would be approximately 0.12g (corresponding to a 72-year return period); and the PGA_{II} value for EQ-II would be 0.145g (corresponding to a 950 year return period). These PGA values for EQ-I and EQ-II are not very different in this example because the example site has relatively low seismicity and the three sources have low maximum magnitudes.

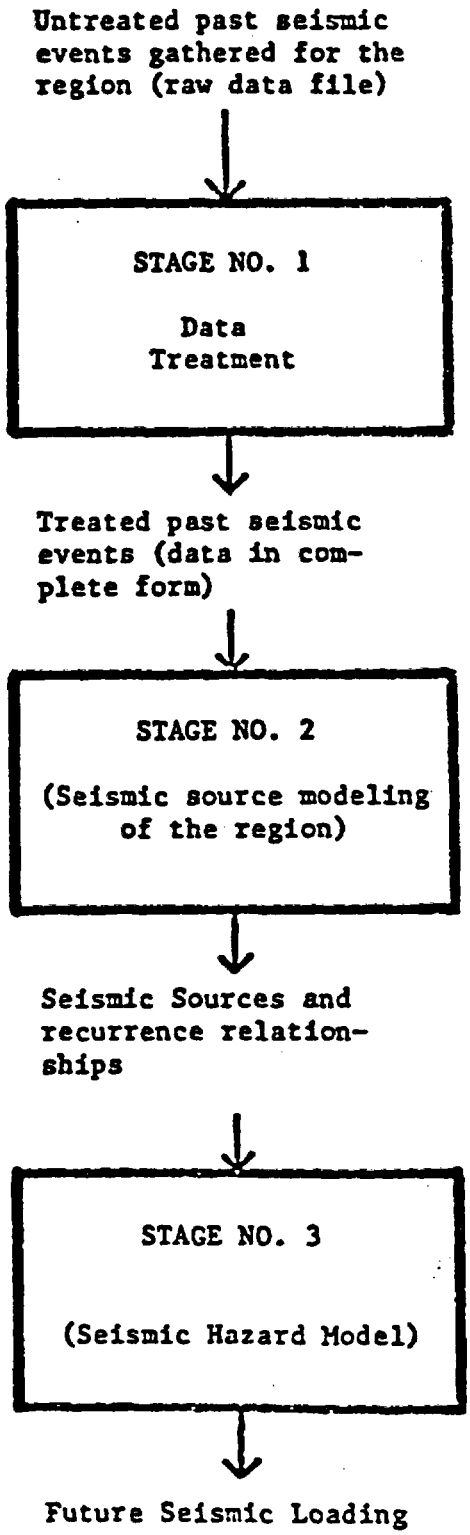


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Figure D-7. Scheme of present seismic hazard methodology.

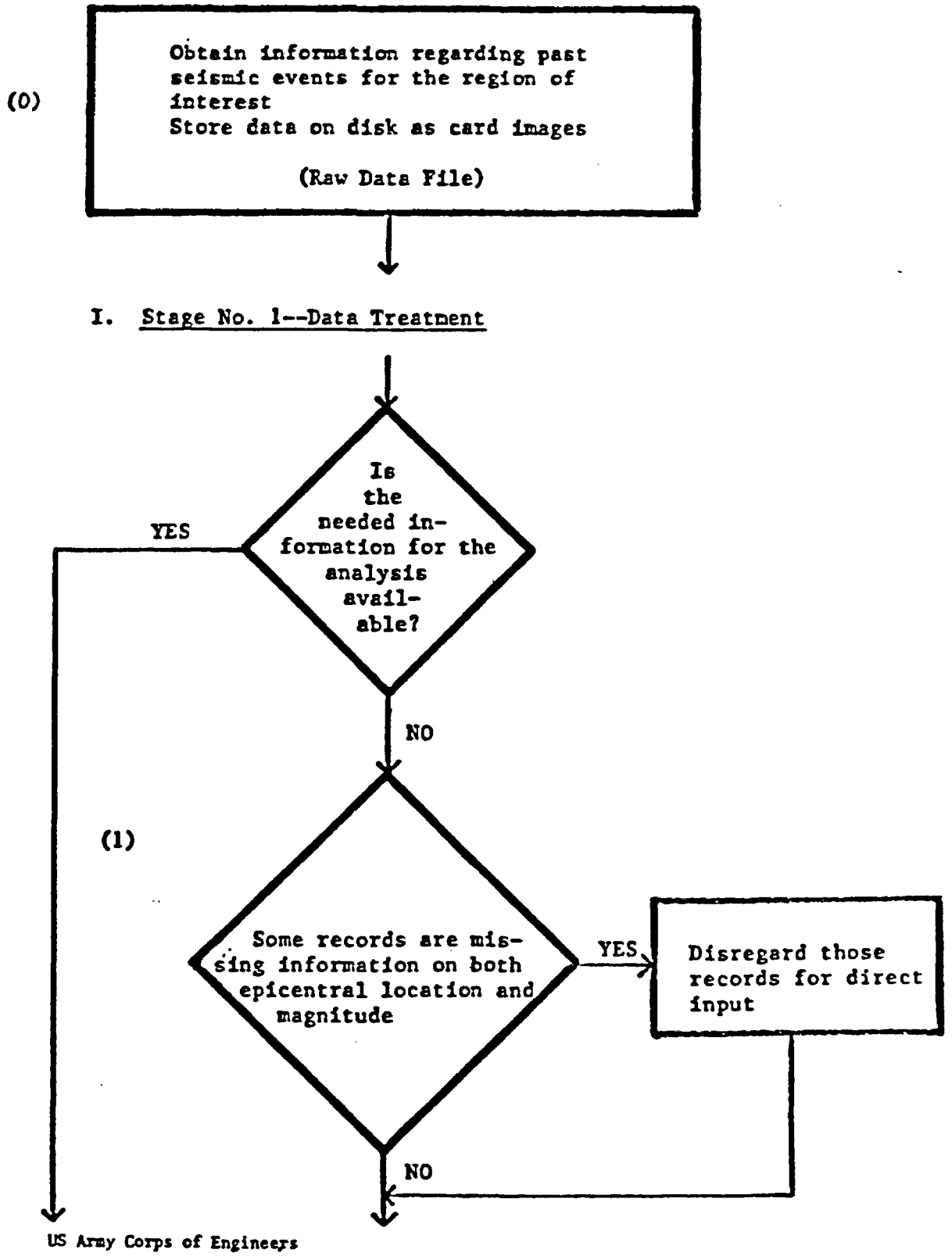


Figure D-8. General flow chart for seismic hazard analysis.

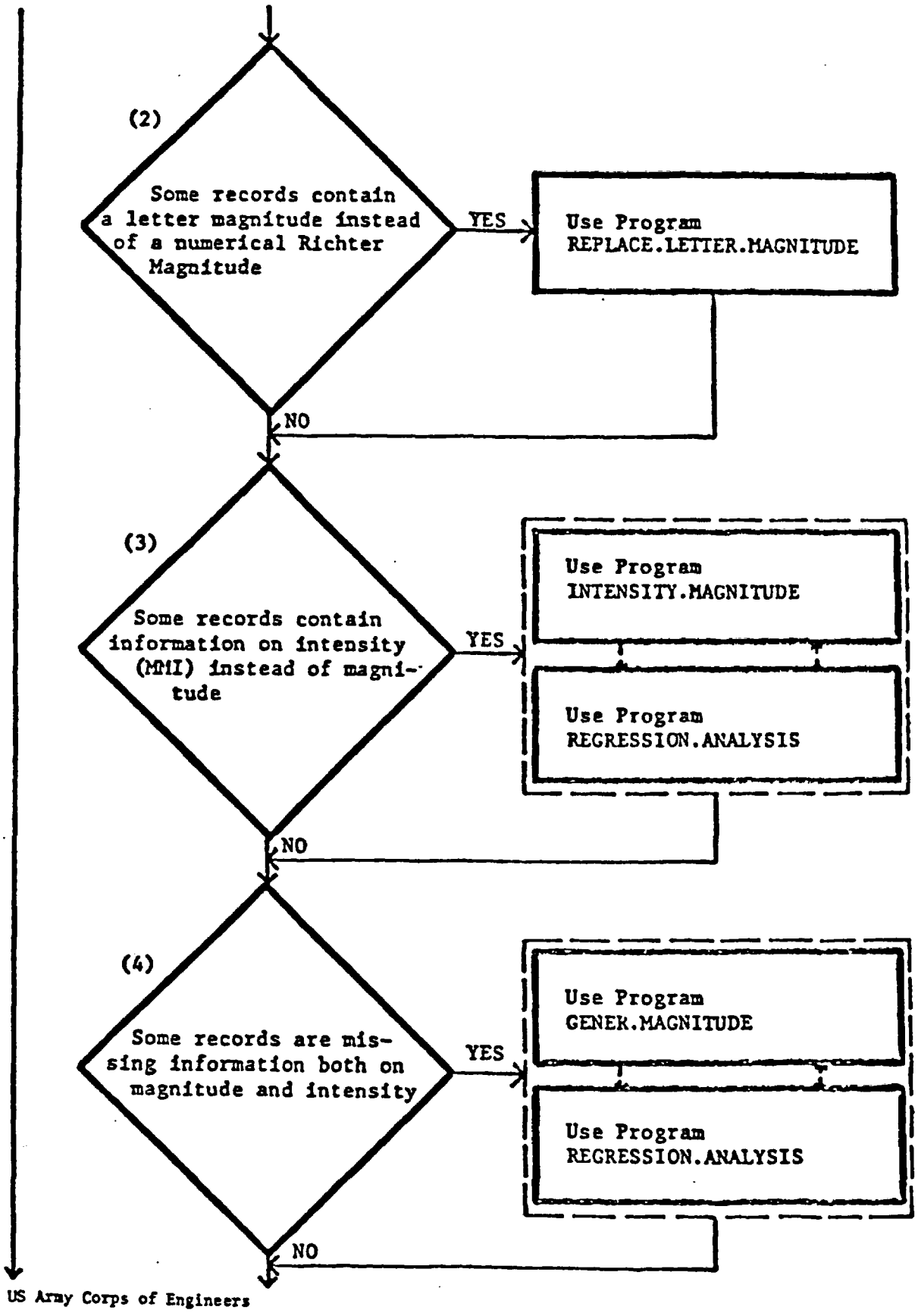
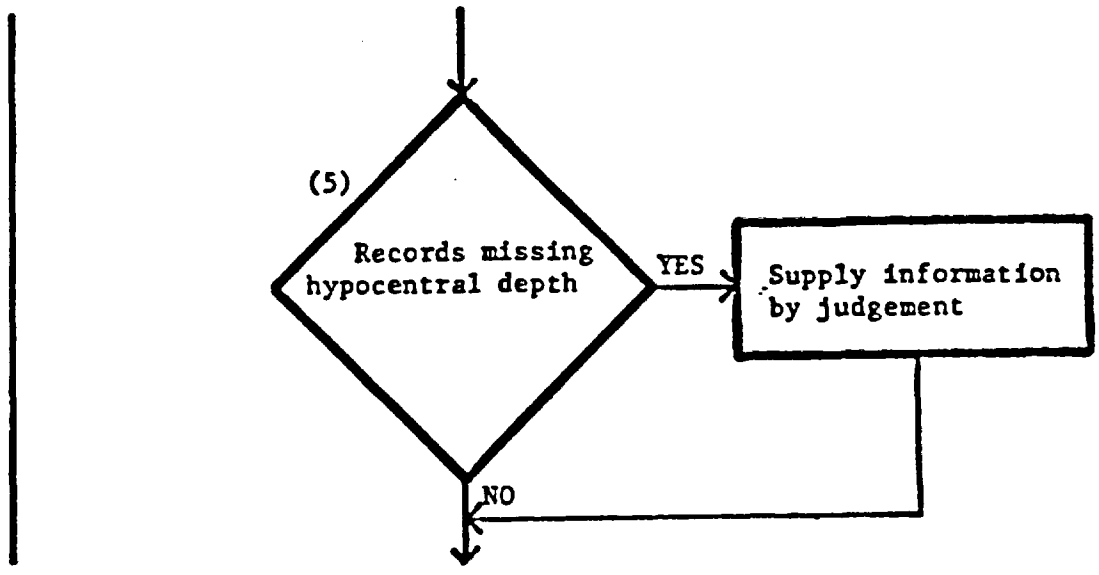
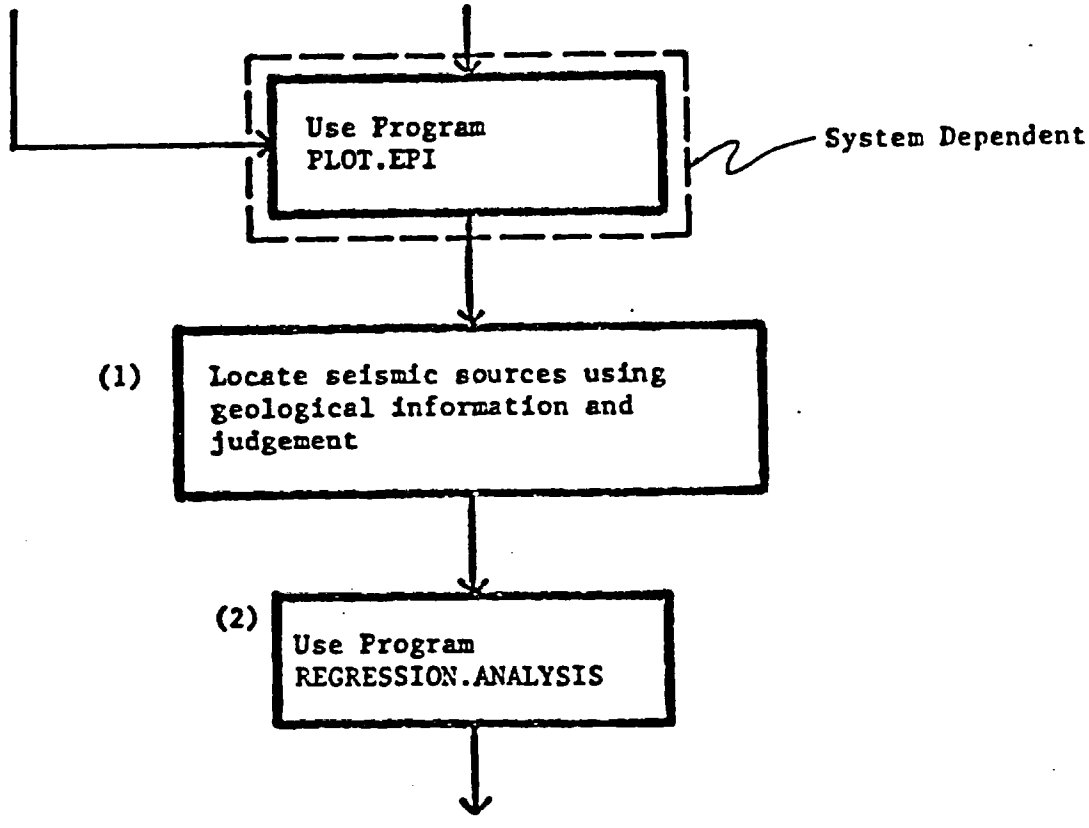


Figure D-8. General Flow Chart for Seismic Hazard Analysis—continued.



II. Stage No. 2--Seismic Modeling of the Region



III. Stage No. 3--Seismic Hazard Model

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Figure D-8. General Flow Chart for Seismic Hazard Analysis—continued.

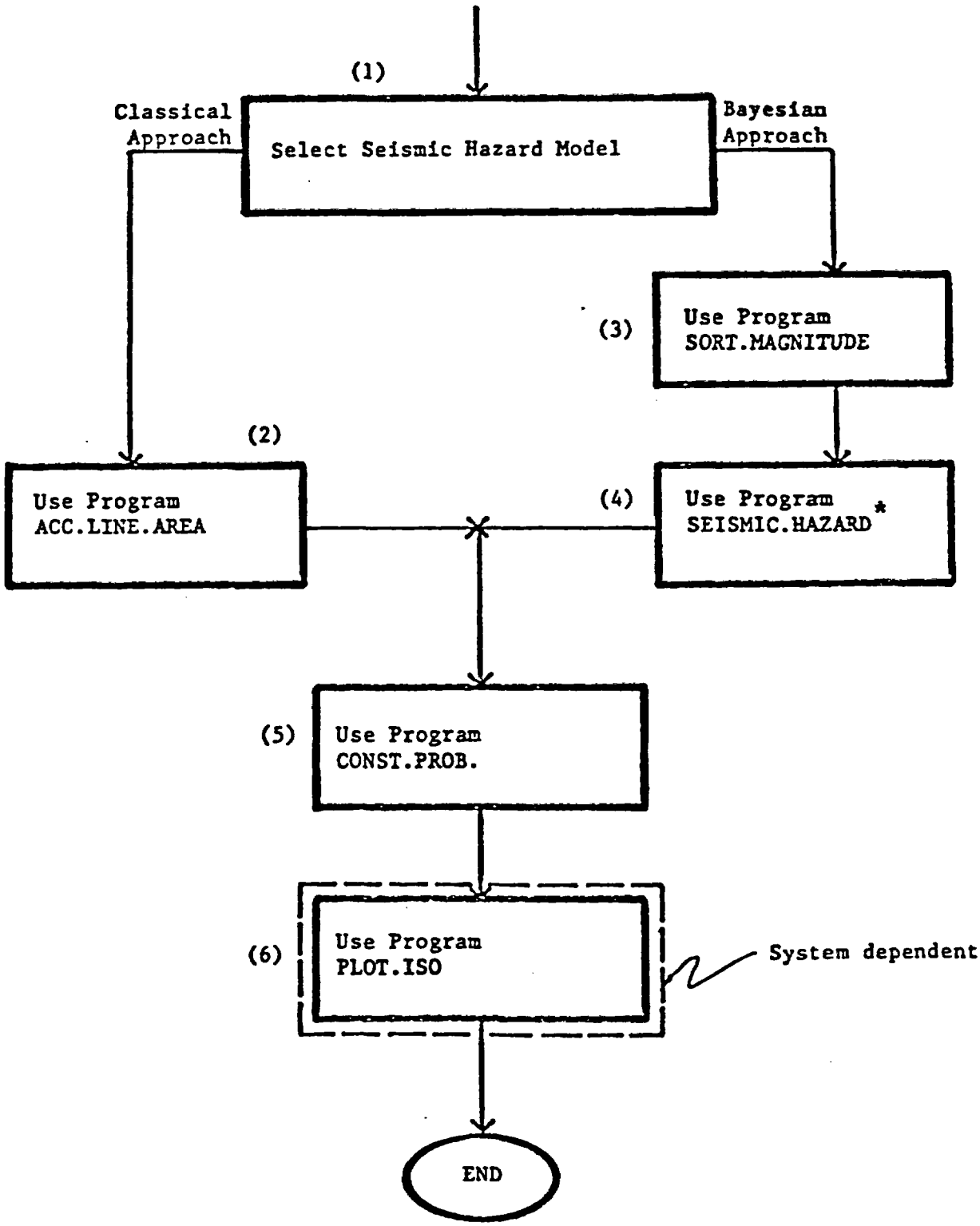

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Figure D-8. General Flow Chart for Seismic Hazard Analysis—continued.

SOURCE	DAY	MONTH	YEAR	HOUR	MINUTE	LATITUDE	LONGITUDE	CLASS	MMI	RADIUS	DEPTH	MS	MB	MR1	MR2	SYMBOL
A.GRAN	17	12	1850	12	30	36.500N	07.400E		6.5					4.61	4.61	I
A.GRAN	17	06	1903	00	24	36.500N	07.500E		7.5					5.25	5.25	I
A.GRAN	04	08	1903	02	11	36.400N	06.600E	D	8.0					5.10	5.10	
A.GRAN	03	12	1928	05	30	36.400N	07.200E	D						5.00	5.00	
A.GRAN	10	02	1937	16	16	36.400N	07.500E	D	9.0					5.40	5.40	
A.GRAN	05	08	1947	09	46	36.300N	06.667E	D	8.5					5.30	5.30	
A.GRAN	27	10	1947	10	29	37.600N	08.500E	D	5.5						5.40	L
A.GRAN	22	11	1950	02	43	36.100N	07.200E	E	5.0						4.10	L
A.GRAN	01	04	1952	04	21	36.500N	07.300E	E	6.0					4.50	4.50	
A.GRAN	12	04	1952	16	23	36 500N	07.300E	E	5.5					4.20	4.20	
A.GRAN	23	05	1956	06	37	36.400N	07.300E	E	7.5						5.00	L
A.GRAN	26	06	1956	01	50	36.000N	08.100E	E	7.0						4.15	L
A.GRAN	02	09	1958	12	26	36.500N	07.400E	F	5.0						3.55	L
A.GRAN	14	11	1959	16	10	36.400N	07.500E	F	4.5						3.05	L
A.GRAN	05	03	1960	04	18	36.600N	07.100E	F	5.5						4.00	L
A.GRAN	02	12	1961	12	40	36.500N	08.200E							5.50	5.50	
A.GRAN	14	03	1963	12	25	36.200N	06.100E	E	7.0						4.40	L
A.GRAN	14	04	1967	23	44	36 500N	07.800E	E						4.30	4.30	

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Figure D-9. Earthquake listing for example 2.

```
REGRESSION ANALYSIS
LINEAR-LN SCALE      'ERC    AREA       RMOX
SAMPLE PROBLEM 1      18     0.0       4.200

NUMBER OF RECORDS INCLUDED    18
AREA                    0.0
TIME (YEARS)        125.00
MINIMUM MAGNITUDE                    3.00
MAGNITUDE INCREMENT FOR CDF          0.20
EARTHQUAKE MAGNITUDES
      4.60      5.20      3.10      5.00      3.40      3.30      3.40      4.10      4.50      4.20      3.00      4.10
      3.50      3.00      4.00      3.50      4.40      4.30
       RM          INTERVAL          CUMULATIVE FREQUENCY
     INTERVAL     FREQUENCY          OCCURRENCES ABOVE RM

   3.00 - 3.19       1                    18.          3.00
   3.20 - 3.39       0                    17.          3.20
   3.40 - 3.59       1                    17.          3.40
   3.60 - 3.79       0                    16.          3.60
   3.80 - 3.99       0                    16.          3.80
   4.00 - 4.19       3                    16.          4.00
   4.20 - 4.39       2                    13.          4.20
   4.40 - 4.59       2                    11.          4.40
   4.60 - 4.79       1                     9.          4.60
   4.80 - 4.99       0                     8.          4.80
   5.00 - 5.19       3                     8.          5.00
   5.20 - 5.39       2                     5.          5.20
   5.40 - 5.59       3                     3.          5.40
TWO STRAIGHT LINES WILL BE USED TO FIT THE DATA
BREAK POINT MAGNITUDE    4.20
    7 POINTS IN THE FIRST LINE
    7 POINTS IN THE SECOND LINE
INTERCEPT AND SLOPE OF LINE    1
 STATISTICS FORM REGRESSION LINE SEGMENT =  1
X-MEAN=    3.59999     Y-MEAN=    2.77707     XVAR=    0.16003     YVAR=    0.00910
COVARXY=   -0.03307     COEFF. OF VAR.=    0.74502     VAR(LNRM)=    0.00328     STDV(LNRM)=    0.05723

ALPHA  3.521115
BETA  -0.206679
INTERCPT AT 3.          5.        6.        7.
        18.19374  12.83300   9.76686   7.95946
INTERCEPT AND SLOPE OF LINE    2
 STATISTICS FORM REGRESSION LINE SEGMENT =  2
X-MEAN=    4.79999     Y-MEAN=    2.00306     XVAR=    0.16003     YVAR=    0.21342
COVARXY=   -0.17418     COEFF. OF VAR.=    0.88759     VAR(LNRM)=    0.03361     STDV(LNRM)=    0.18334

ALPHA  7.225758
BETA  -1.087904
INTERCPT AT 3.          5.        6.        7.
        52.56635   3.96714   2.81046   0.67737
INTERSECTION POINT
MAGNITUDE    4.20
LN OF N      2.65
```

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Figure D-10. Output for recurrence relationship, example 2.

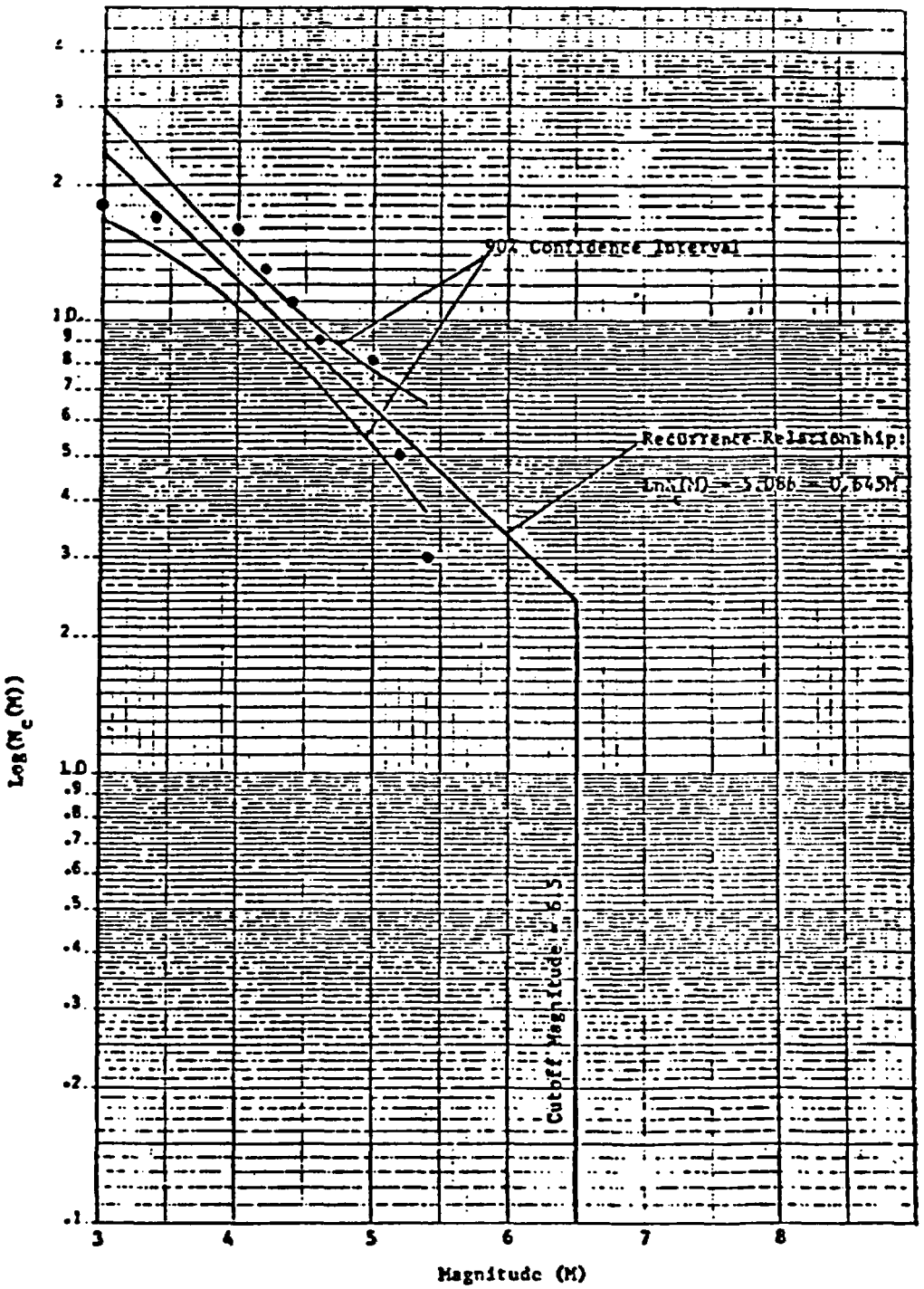

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Figure D-11. Recurrence relationship for example 2.

```
REGRESSION ANALYSIS
LINEAR-LN SCALE      NBRC     AREA       RMDK
SAMPLE PROBLEM 1      16      0.0        0.0

NUMBER OF RECORDS INCLUDED    16
AREA                 0.0
TIME (YEARS)       125.00
MINIMUM MAGNITUDE                    3.00
MAGNITUDE INCREMENT FOR CDF          0.20
EARTHQUAKE MAGNITUDES
      4.60      5.20      5.10      5.00      5.40      5.30      5.40      4.10      4.50      4.20      5.00      4.10
      3.50      3.00      4.00      5.50      4.40      4.30
       RM          INTERVAL          CUMULATIVE FREQUENCY
    INTERVAL      FREQUENCY          OCCURRENCES ABOVE RM

   3.00 - 3.19       1                   16.          3.00
   3.20 - 3.39       0                   17.          3.20
   3.40 - 3.59       1                   17.          3.40
   3.60 - 3.79       0                   16.          3.60
   3.80 - 3.99       0                   16.          3.80
   4.00 - 4.19       3                   16.          4.00
   4.20 - 4.39       2                   13.          4.20
   4.40 - 4.59       2                   11.          4.40
   4.60 - 4.79       1                    9.          4.60
   4.80 - 4.99       0                    8.          4.80
   5.00 - 5.19       3                    8.          5.00
   5.20 - 5.39       2                    5.          5.20
   5.40 - 5.59       3                    3.          5.40
INTERCEPT AND SLOPE OF LINE    1
 STATISTICS FORM REGRESSION LINE SEGMENT =  1
X-MEAN=    4.19999     Y-MEAN=    2.37704     XVAR=    0.56004     YVAR=    0.27831
COVARXY=   -0.36135     COEFF. OF VAR.=    0.83776     VAR(LNRM)=    0.05336     STDV(LNRM)=    .23100

ALPHA  5.036937
BETA  -0.645227
INTERCPT AT 3.        5.        6.        7.
        23.36659   6.42923   3.37241   1.76098
   90 %  CONFIDENCE INTERVALS

 CONF. VALUE=    2.20098     ERROR INDIC.=     0

X =        3.000     3.200     3.400     3.600     3.800     4.000     4.200     4.400     4.600     4.800
UPCNF=    30.502    25.987    22.190    19.009    16.363    14.103    12.404    10.957     9.765     8.764
DNCNF=    17.930    16.231    14.605    13.243    11.885    10.592     9.356     8.183     7.093     6.105
X =        5.000     5.200     5.400
UPCNF=     7.903     7.150     6.453
DNCNF=     5.230     4.466     3.805
```

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Figure D-12. Output for bilinear recurrence relationship, example 2.

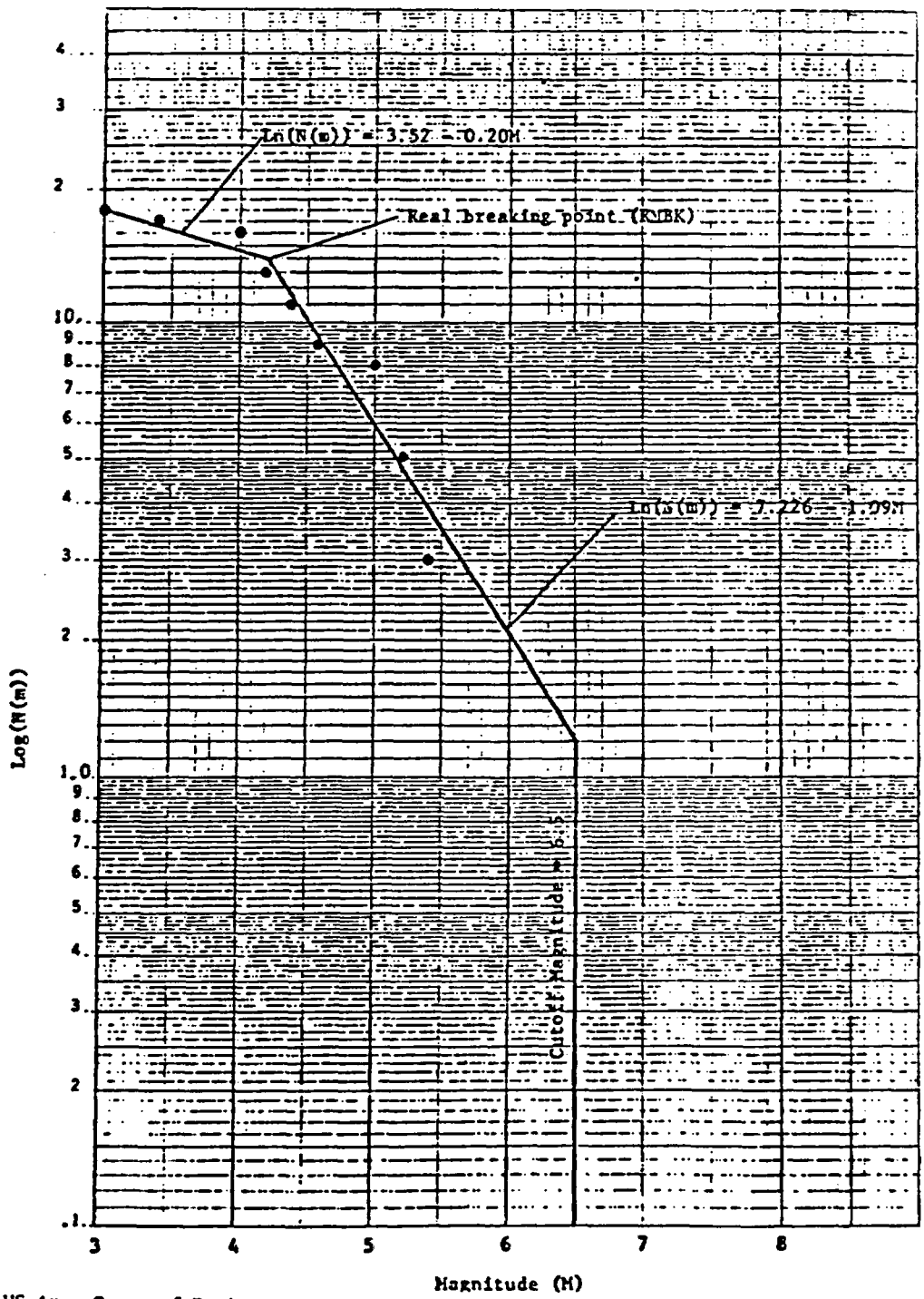

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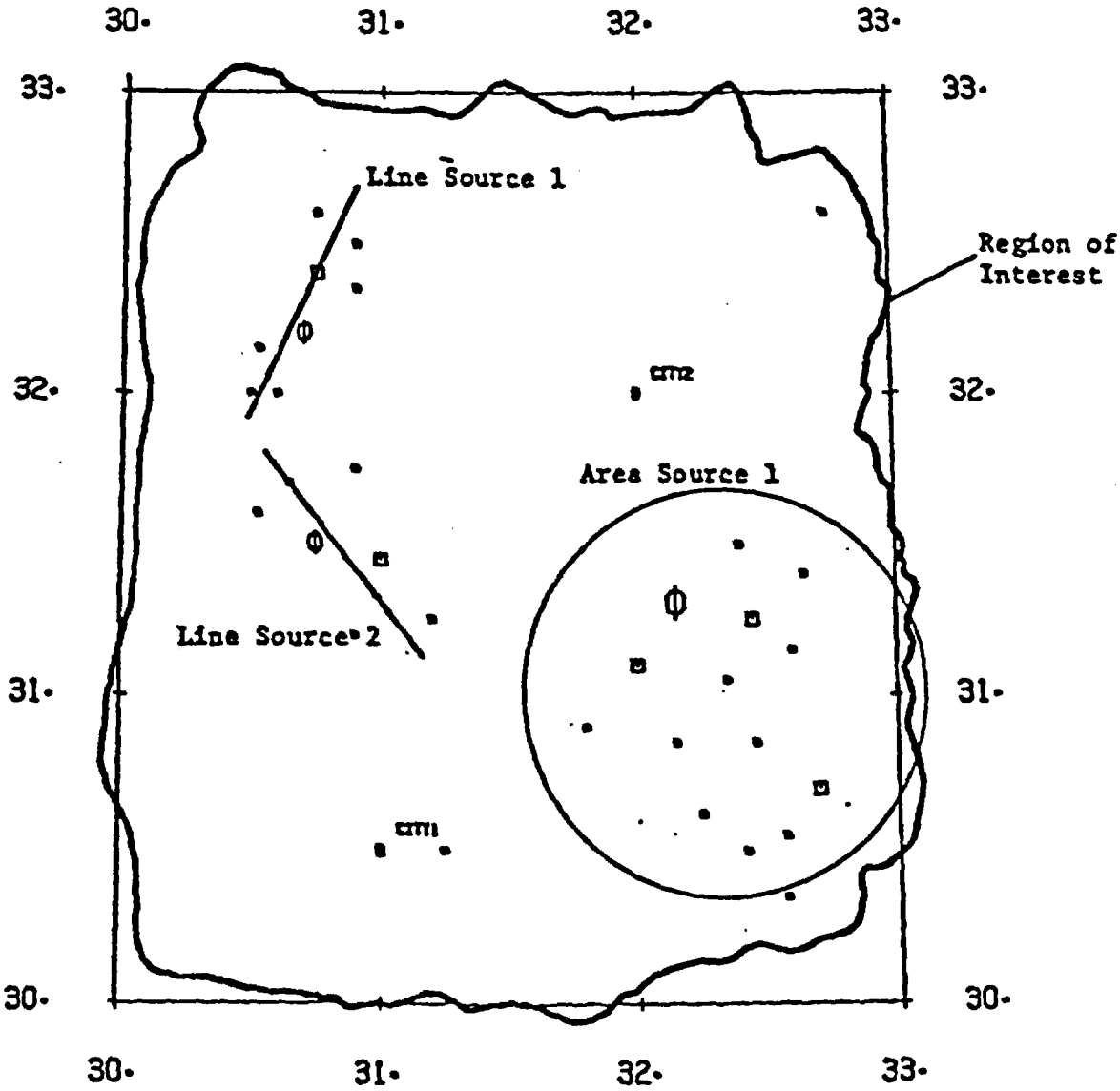
Figure D-13. Bilinear recurrence relationship for example 2.



o Magnitude 4+
o Magnitude 5+
ø Magnitude 6+
ø Magnitude 7+
Plot of epicenters (sample problem)

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Figure D-14. Seismic sources for region of example 3.

EARTHQUAKE DATA SORTED BY SOURCES

SOURCE	DAY	MONTH	YEAR	HOUR	MINUTE	LATITUDE	LONGITUDE	CLASS	MMI	RADIUS	DEPTH	MS	MB	MR1	MR2	SYMBOL
LINE SOURCE 1 (8 RECORDS)																
A.GRAN	02	03	1900	07	00	32.600N	30.750E							3.50	3.50	
A.GRAN	03	05	1902	05	00	32.500N	30.900E							3.75	3.75	
A.GRAN	05	03	1905	01	00	32.350N	30.900E							4.75	4.75	
A.GRAN	05	03	1912	02	00	32.000N	30.500E							3.25	3.25	
A.GRAN	08	01	1920	02	30	32.200N	30.700E							6.00	6.00	
A.GRAN	04	03	1965	09	30	32.000N	30.600E							4.65	4.65	
A.GRAN	03	03	1973	03	30	32.400N	30.750E							5.00	5.00	
A.GRAN	06	04	1976	05	00	32.150N	30.530E							3.25	3.25	
LINE SOURCE 2 (9 RECORDS)																
A.GRAN	04	09	1916	01	00	31.600N	30.530E							3.50	3.50	
A.GRAN	06	03	1921	09	10	31.700N	30.650E							4.50	4.50	
A.GRAN	04	01	1935	15	30	31.450N	31.000E							5.55	5.55	
A.GRAN	10	03	1937	01	15	31.200N	30.900E							3.60	3.60	
A.GRAN	04	12	1940	03	00	31.250N	31.200E							4.10	4.10	
A.GRAN	12	01	1972	11	05	31.750N	30.900E							4.65	4.65	
A.GRAN	11	05	1975	01	15	31.500N	30.750E							6.30	6.30	
A.GRAN	01	03	1976	03	12	30.500N	31.250E							3.50	3.50	
A.GRAN	01	07	1978	03	15	30.500N	32.420E							4.25	4.25	
AREA SOURCE 1 (15 RECORDS)																
A.GRAN	17	02	1923	06	00	31.050N	32.350E							4.35	4.35	
A.GRAN	16	01	1925	14	00	30.700N	32.700E							5.60	5.60	
A.GRAN	30	11	1925	12	15	31.500N	32.400E							3.50	3.50	
A.GRAN	14	02	1948	01	00	31.250N	32.450E							5.60	5.60	
A.GRAN	13	04	1950	13	30	31.150N	32.600E							3.60	3.80	
A.GRAN	18	11	1951	02	15	31.300N	32.150E							7.00	7.00	
A.GRAN	15	06	1954	06	35	31.100N	32.000E							5.60	5.60	
A.GRAN	02	12	1958	06	15	30.900N	31.800E							3.00	3.00	
A.GRAN	18	01	1960	04	18	30.620N	32.250E							4.65	4.65	
A.GRAN	01	01	1968	13	14	30.550N	32.570E							3.40	3.40	
A.GRAN	04	10	1969	02	00	30.650N	32.150E							3.15	3.15	
A.GRAN	03	12	1970	10	12	30.350N	32.570E							3.00	3.00	
A.GRAN	17	03	1972	13	05	30.850N	32.460E							4.50	4.50	
A.GRAN	08	11	1973	15	00	32.600N	32.750E							3.50	3.50	
A.GRAN	16	10	1976	10	00	31.400N	32.650E							3.65	3.65	

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Figure D-15. Earthquake listing for sources in example 3.

```
PROGRAM ACC.LINE.AREA (SAMPLE PROBLEM)
ATTENUATION CONSTANTS

B1= 0.42993700-03   B2= 0.80000000+00   B3= 0.20000000+01   B4= 0.36737690+00
DELTAL = 0.50000000-01   DELTAC = 0.30000000-01

TIME PERIODS
     50.00
ACCELERATIONS
      0.05      0.10      0.15      0.20      0.25      0.30      0.35      0.40      0.45      0.50
      0.55      0.60      0.65      0.70      0.75      0.80
LINE SOURCES
************

LINE SOURCE 1
  ALPHA1          BETA1           XL1           XL2           YL1           YL2           WL
  0.25600D+01  -0.10900D+01   0.30500D+02   0.30920D+02   0.31970D+02   0.32620D+02   0.87000D-01
SECOND REGRESSION CONSTANTS
  ALPHAL2         BETAL2          MR
  0.24000D+02  -0.45500D+01   0.64500D+01   0.68000D+01
LINE SOURCE 2
  ALPHA1          BETA1           XL1           XL2           YL1           YL2           WL
  0.31700D+01  -0.74000D+00   0.30510D+02   0.31300D+02   0.31750D+02   0.31000D+02   0.87000D-01
SECOND REGRESSION CONSTANTS
  ALPHAL2         BETAL2          MR
  0.79150D+02  -0.12400D+02   0.65000D+01   0.78000D+01


AREA SOURCES
************

AREA SOURCE 1
  ALPHA1          BETA1           XO            YO            W             WA
  0.14000D+00  -0.70000D-01   0.32390D+02   0.31078D+02   0.74900D+00   0.87000D-01
SECOND REGRESSION CONSTANTS
  ALPHA2          BETA2           MR
  0.79900D+02  -0.13040D+02   0.61500D+01   0.65000D+01
********** P R O B A B I L I T Y   D I S T R I B U T I O N   O F   P E A K   G R O U N D   A C C E L E R A T I O N
***

SITE OF INTEREST (CITY 2)
GEOMETRIC CONSTANTS

NL=    2    NA =    1    NOMAX=    1    NYMAX=    1    NT=    1

SITE LOCATION
X=    32.000    Y=    32.060
TIME PERIOD =  50.00 YRS
PGA =      0.0500    0.1000    0.1500    0.2000    0.2500    0.3000    0.3500    0.4000    0.4500    0.5000
P(Y>YO)    1.0000    0.7512    0.0043    0.0       0.0       0.0       0.0       0.0       0.0       0.0
P(Y<YO)    0.0000    0.2488    0.9957    1.0000    1.0000    1.0000    1.0000    1.0000    1.0000    1.0000
PGA =      0.5500    0.6000    0.6500    0.7000    0.7500    0.8000
P(Y>YO)    0.0       0.0       0.0       0.0       0.0       0.0
P(Y<YO)    1.0000    1.0000    1.0000    1.0000    1.0000    1.0000
```

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Figure D–16. Output for recurrence relationships and site PGA probability distribution for example 3.

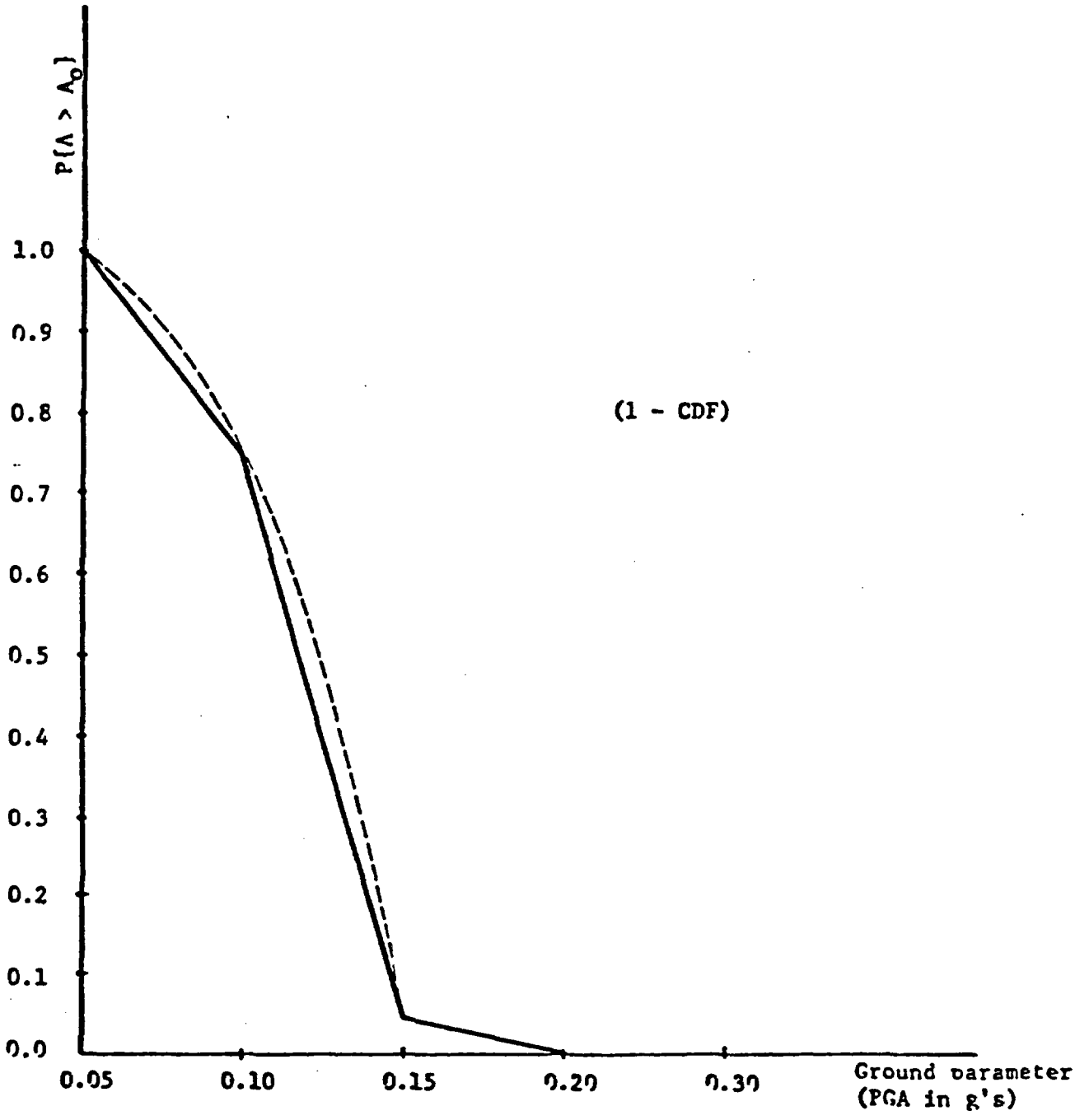

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Figure D-17. Complementary cumulative distribution function for example 3.

Table D-1. Return period vs. PGA for CITY 2.

PGA in g units	Return Period in Years
0.06	18
0.075	23
0.100	37
0.110	63
0.120	87
0.130	141
0.140	358
0.150	10000

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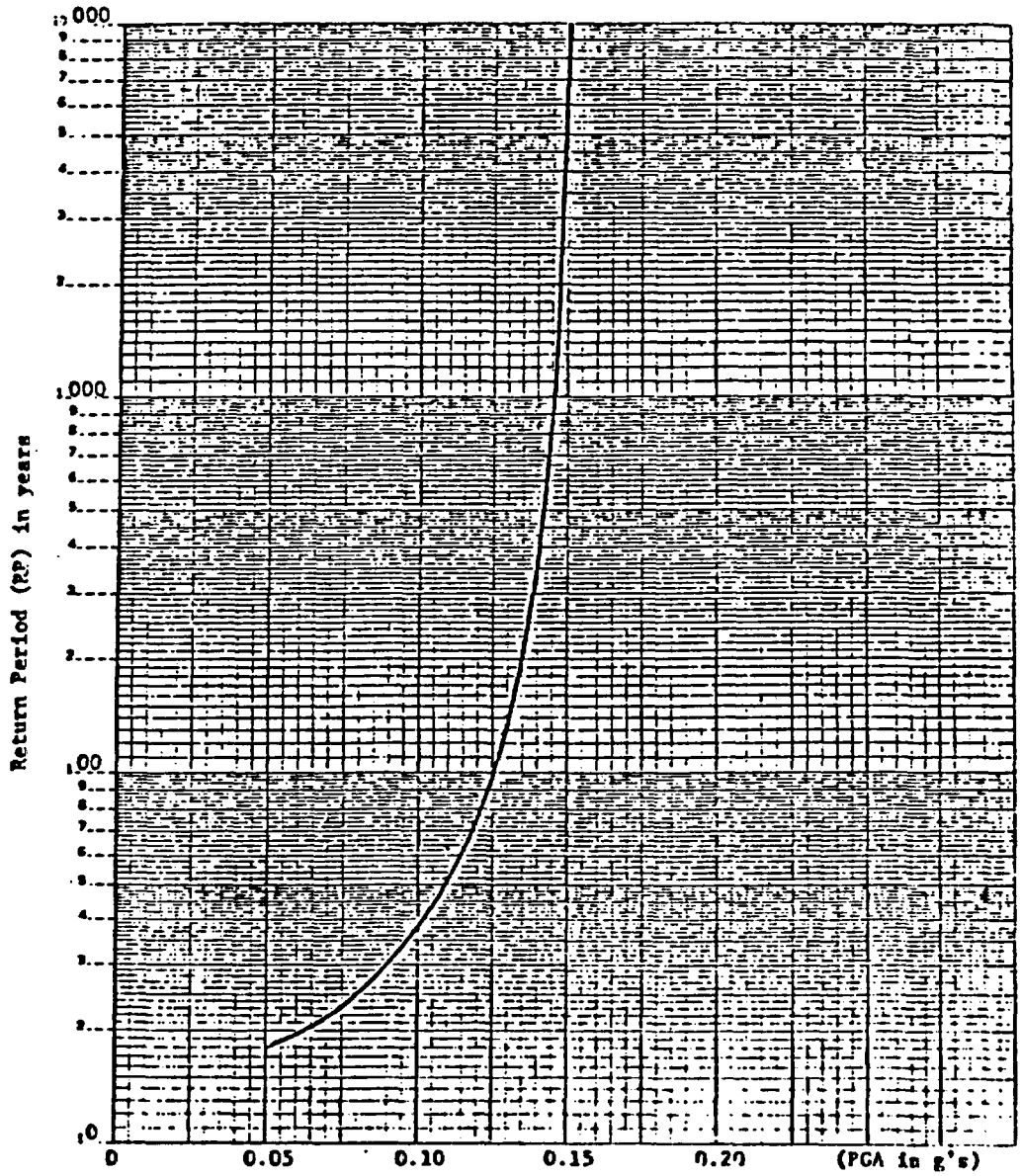

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Figure D-18. Acceleration zone graph (AZG) for CITY 2.

APPENDIX E
DESIGN EXAMPLES—STRUCTURES

E-1. Purpose and scope.

This appendix gives illustrative examples for designing and analyzing various types of lateral systems in accordance with the criteria and procedures of chapters 4 and 5 of this manual.

E-2. Use of appendix.

The design examples are purely advisory; they are not intended to place super-restrictions on the manual. This appendix is not a handbook for the inexperienced designer. Neither the manual or the manual supplemented by the appendices can replace good engineering judgment in specific situations. Designers are urged to study the entire manual.

Table E-1. Design Examples—Structures

Fig. No.	*Example No. and Description*
E-1	E-1 Sample modal analyses.
E-2	E-2 Box system. A 2-story building with bearing walls in concrete using a series of interior, vertical-load-carrying columns and girder bents.
E-3	E-3 Steel ductile moment-resisting space frame and steel braced frame. A 3-story building with transverse ductile moment-resisting frames and longitudinal frames with K-bracing.
E-4	E-4 Concrete ductile moment-resisting space frame. A 7-story building with a complete ductile moment-resisting space frame in concrete without shear walls.

DESIGN EXAMPLE: E-1

SAMPLE MODAL ANALYSES:

Purpose. This example is presented to illustrate the method of obtaining story forces, accelerations, and displacements from given building characteristics and ground motion response spectra. The results are shown in a format similar to the sample format used in the equivalent static force procedure of the Basic Design Manual, table 4-4. Thus, a comparison of static force procedures and dynamic analysis procedures can be made. The data in this example serve as a back-up for the examples given in paragraph 2-5c of this manual. The results are graphically displayed in figures 2-9 and 2-10 of this manual.

Description of Structure. The data on sheets 3 through 6 are based on the characteristics of a 7-story reinforced concrete moment-resisting space frame building. Sheet 7 represents a 30-story building. The model for this building was developed by expanding the 7-story building characteristics. Each story mass (w/g) of the 30-story building lumped mass model was assumed to represent 4 stories similar to those of the 7-story building (i.e., the indicated story plus one-and-one-half stories above and below). This was done only for illustrative purposes to demonstrate the influences of higher modes of vibration for taller buildings with longer periods of vibration (refer to para 2-5c(3)).

Response Spectrum. The modal analyses were performed on the basis of the 5-percent damped response spectrum shown in figure 2-8 of this manual.

Masses, Mode Shapes, and Periods. Story masses were obtained from the calculated story weights of the building. A mathematical model of the building was developed from the section properties of the structural system. The building was modeled as a series of two-dimensional frames. A computer program that analyzes two-dimensional framing systems was used to determine the periods and mode shapes of the first three modes of vibration. In this computer program, each mode is normalized for $\Sigma(w/g)\phi^2 = 1.0$. The mode shapes are shown in figure 2-6 of this manual. In figure 2-6, the modes are normalized to a value of 1/2-inch at the top story.

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Example E-1	1 of 7	Sample Modal Analyses

Figure E-1. Sample modal analysis.

Modal Analysis to Determine Total Base Shear and Story Accelerations. Sheet 3 illustrates a hand-calculation procedure to determine the total base shear and the story accelerations using mass, mode shape, period, and response spectrum data. Equations 4-1 and 4-2 are used to determine the participation factors. The spectral acceleration (S_a) for the period (T) of each mode is determined from the response spectrum. The story accelerations (a) are determined from equation 6-1 and the base shears (V) are determined from equation 4-4. The sum of the participation factors (P.F. and α) add up to 1.08 and 0.986, respectively. These values being close to the value of 1.0 indicate that most of the model participation is included in the three modes considered in this example (refer to paras 4-3c(1)(b) and 5-4c(2)). The story accelerations and the base shears are combined by the square-root-of-the-sum-of-the-squares (SRSS) on the last column of the table. The modal base shears are 2408 kips, 632 kips, and 200 kips for the first, second, and third modes, respectively. These are used on the following sheets to determine story forces. The SRSS base shear is 2498 kips.

Story Forces, Accelerations, and Displacements. Sheets 4, 5, and 6 are set up in a manner very similar to the Basic Design Manual, table 4-4. In the static lateral force procedure, $wh/\Sigma wh$ is used to distribute the force on the assumption of a straight line mode shape. In the dynamic analysis, the more representational $w\phi/\Sigma w\phi$ is used to distribute the forces for each mode. Story shears and overturning moments are determined in the same manner for each method. Modal story accelerations are determined by dividing the story force by the story weight. These are essentially the same values as shown on sheet 3 (slight differences are due to rounding off). The SRSS of the accelerations of sheet 3 are roughly estimated in the static procedure by the bracketed quantity in equation 3-9 of the Basic Design Manual and are listed in the last column of table 4-4 in that manual. Modal story displacements (δ) are calculated from the accelerations and the period (equations 4-5 and 6-1 of this manual). Modal interstory drifts ($\Delta\delta$) are calculated by taking the differences between the δ values of adjacent stories. The values shown on sheets 4, 5, and 6 of this design example are summarized in table 5-3 and are plotted with the SRSS combination in figure 2-10.

Thirty-Story Example. Sheet 7 shows the model analysis for base shears and story accelerations for the 30-story example. This parallels the 7-story example on sheet 3. Parallel tables for sheets 4, 5, and 6 are not shown, but the results are summarized in figure 2-9.

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Example E-1	2 of 7	Sample Modal Analyses

Figure E-1. Sample modal analysis—continued.

7 STORY REINFORCED CONCRETE FRAME BUILDING

Mathematical Model - Gross Concrete Sections, Spandrel Beams increased 50% for Slab participation.

Periods (T) and Mode Shapes (ϕ) have been calculated by a two-dimensional computer program.

Spectral Accelerations for 3 modes obtained from Response Spectrum

GIVEN: Masses (w/g), ϕ's, T's, S_a's.

REQ'D: Story Accelerations (a) and Base Shear Forces (V)

LEVEL	$\frac{w}{g}$ $\left(\frac{k\text{-}s^2}{ft}\right)$	MODE 1				MODE 2				MODE 3				SRSS
		ϕ_1	$\frac{w}{g}\phi_1$	$\frac{w}{g}\phi_1^2$	a_1 (g)	ϕ_2	$\frac{w}{g}\phi_2$	$\frac{w}{g}\phi_2^2$	a_2 (g)	ϕ_3	$\frac{w}{g}\phi_3$	$\frac{w}{g}\phi_3^2$	a_3 (g)	a_x (g)
Roof	43.78	.0794	3.48	.276	0.362	.0747	3.27	0.244	-0.235	.0684	2.99	0.205	0.120	0.448
7	45.34	.0745	3.38	.252	0.340	.0411	1.86	0.076	-0.129	-.0040	-.18	0.001	-0.007	0.364
6	45.34	.0666	3.02	.201	0.304	-.0042	-.19	0.001	0.013	-.0644	-2.92	0.188	-0.113	0.325
5	45.34	.0558	2.53	.141	0.254	-.0471	-2.14	0.101	0.148	-.0630	-2.86	0.180	-0.111	0.314
4	45.34	.0425	1.93	.082	0.194	-.0718	-3.26	0.234	0.226	-.0023	-.10	0.000	-0.004	0.298
3	45.34	.0279	1.27	.035	0.127	-.0697	-3.16	0.220	0.219	.0604	2.74	0.166	0.106	0.275
2	56.83	.0149	0.85	.013	0.068	-.0467	-2.65	0.124	0.147	.0677	3.85	0.261	0.119	0.201
1	—	0	0	0	0	0	0	0	0	0	0	0	0	0
Σ	327.31		16.46	1.000			-6.27	1.000			3.52	1.001		
PF_{roof}	(Eq. 4-1)	$\frac{16.46}{1.000}(.0794) = 1.31$				$\frac{-6.27}{1.000}(.0747) = -0.47$				$\frac{3.52}{1.001}(.0684) = 0.24$				$\Sigma = 1.08$
α	(Eq. 4-2)	$\frac{(16.46)^2}{(327.31)(1.000)} = 0.828$				$\frac{(-6.27)^2}{(327.31)(1.000)} = 0.120$				$\frac{(3.52)^2}{(327.31)(1.001)} = 0.038$				$\Sigma = .986$
T		0.880 sec.				0.288 sec.				0.164 sec.				
S_a		0.276 g				0.500 g				0.500 g				
a_{comp}	(Eq. 6-1)	$(1.31)(0.276) = 0.362$ g				$(-0.47)(0.500) = -0.235$ g				$(0.24)(0.500) = 0.120$ g				0.448
V	(Eq. 4-4)	$(.828)(.276)(10,539) = 2,408$ kips				$(0.12)(.500)(10,539) = 632$ kips				$(.038)(.500)(10,539) = 200$ kips				2498 k
V/W		0.229				0.060				0.019				0.237

$W = \Sigma\left(\frac{w}{g}\right) \cdot g = 327.31 \times 32.2 = 10,539$ kips Building Weight

$A_G = 0.20$ g Site PGA

$\beta = .05$ Damping Factor

MODAL ANALYSIS TO DETERMINE TOTAL BASE SHEAR & STORY ACCELERATIONS

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Example E-1 3 of 7 Sample Modal Analyses

Figure E-1. Sample modal analysis—continued.

SEVEN STORY BLDG. - TRANSVERSE DIRECTION

1ST MODE FORCES $T_1 = 0.880$ sec.

MODAL BASE SHEAR $V_1 = 2408^K$

(1) STORY	(2) ϕ	(3) h FT.	(4) Δh FT.	(5) ω KIPS	(6) $\frac{\omega\phi}{\Sigma\omega\phi}$	(7) F KIPS $(V_1)\times(6)$	(8) V KIPS $\Sigma(7)$	(9) ΔOTM K-FT $(4)\times(8)$	(10) OTM K-FT $\Sigma(9)$	(11) ACCEL. g $(7)\div(5)$	δ^* FT.	$\Delta\delta$ FT
ROOF	.0794	65.7		1410	0.211	508			0	0.360	.228	
			8.7				508	4420				.014
7	.0745	57.0		1460	0.205	494			4420	0.338	.214	
			8.7				1002	8717				.022
6	.0666	48.3		1460	0.184	443			13,137	0.303	.192	
			8.7				1445	12,572				.031
5	.0558	39.6		1460	0.154	371			25,709	0.254	.161	
			8.7				1816	15,799				.039
4	.0425	30.9		1460	0.117	282			41,508	0.193	.122	
			8.7				2098	18,253				.042
3	.0279	22.2		1460	0.077	185			59,761	0.127	.080	
			8.7				2283	19,862				.057
2	.0149	13.5		1830	0.052	125			79,623	0.068	.043	
			13.5				2408	32,508				.043
GROUND	0	0		0	0	0			112,131	0	0	
				Σ	1.000	2408^K		112,131				

$* \ \delta_{xi} = \frac{g}{4\pi^2} \times T^2 \times F/W = .632 \times \text{Accel.}$

Figure E-1. Sample modal analysis—continued.

SEVEN STORY BLDG. - TRANSVERSE DIRECTION

2ND MODE FORCES $T_2 = .288$ sec.

MODAL BASE SHEAR $V_2 = 632$ KIPS

(1) STORY	(2) ϕ	(3) h FT.	(4) Δh FT	(5) ω KIPS	(6) $\frac{\omega\phi}{\Sigma\omega\phi}$	(7) F KIPS $(V_2)\times(6)$	(8) V KIPS $\Sigma(7)$	(9) ΔOTM K-FT (4)×(8)	(10) OTM K-FT $\Sigma(9)$	(11) ACCEL. g (7)÷(5)	δ^* FT	$\Delta\delta$ FT
ROOF	.0747	65.7		1410	-.522	-330			0	-.234	-.016	
			8.7				-330	-2871				.007
7	.0411	57.0		1460	-.297	-188			-2871	-.129	-.009	
			8.7				-518	-4507				.010
6	-.0042	48.3		1460	.030	19			-7378	.013	.001	
			8.7				-499	-4341				.009
5	-.0471	39.6		1460	.341	216			-11,719	.148	.010	
			8.7				-283	-2462				.005
4	-.0718	30.9		1460	.520	329			-14,181	.225	.015	
			8.7				46	400				.000
3	-.0697	22.2		1460	.504	319			-13,781	.219	.015	
			8.7				365	3176				.005
2	-.0467	13.5		1830	.423	267			-10,605	.146	.010	
			13.5				632	8532				.010
GROUND	0	0							-2073	0	0	
				Σ	.999	632*		-2073				

* $\delta_{x2} = g/4\pi^2 \times T^2 \times F/\omega = .068 \times$ Accel.

Figure E-1. Sample modal analysis—continued.

SEVEN STORY BLDG. - TRANSVERSE DIRECTION

3RD MODE FORCES $T_3 = .164$ SEC.

MODAL BASE SHEAR $V_3 = 200$ KIPS

(1) STORY	(2) ϕ	(3) h FT	(4) Δh FT	(5) ω KIPS	(6) $\frac{\omega\phi}{\Sigma\omega\phi}$	(7) F KIPS $(V_3)(6)$	(8) V KIPS $\Sigma(7)$	(9) ΔOTM K-FT (4)×(8)	(10) OTM K-FT $\Sigma(9)$	(11) ACCEL. g (7)÷(5)	δ^* FT.	$\Delta\delta$ FT.
ROOF	.0684	65.7		1410	0.849	170			0	.121	.003	
			8.7				170	1479				.003
7	-.0040	57.0		1460	-0.051	-10			1479	-.007	.000	
			8.7				160	1392				.003
6	-.0644	48.3		1460	-0.830	-166			2871	-.114	-.003	
			8.7				-6	-52				.000
5	-.0630	39.6		1460	-0.813	-163			2819	-.112	-.003	
			8.7				-169	-1470				.003
4	-.0023	30.9		1460	-0.028	-6			1349	-.004	.000	
			8.7				-175	-1523				.002
3	.0604	22.2		1460	0.778	156			-174	.107	.002	
			8.7				-19	-165				.001
2	.0677	13.5		1830	1.094	219			-339	.120	.003	
			13.5				200	2700				.003
GROUND	0	0							2361	0	0	
				Σ	0.999	200		2361				

$* \ \delta_{x3} = g/4\pi^2 \times T^2 \times F/\omega = .022 \times \text{Accel.}$

Figure E-1. Sample modal analysis—continued.

30 STORY REINFORCED CONCRETE FRAME BUILDING

Mathematical Model - Gross Concrete Sections, Spandrel Beams increased 50% for Slab participation, Masses of Every four stories lumped into one.

Periods (T) and Mode Shapes (Φ) are based on the results of a two-dimensional computer program.

Spectral Accelerations for 3 modes obtained from Resp. Spectrum.

GIVEN: Masses ($\frac{w}{g}$), Φ's, T's, S_a's

REQ'D: Story Accelerations (a) and Base Shear Forces (V)

* LEVEL	w/g $\left(\frac{k \cdot s^2}{ft}\right)$	MODE 1				MODE 2				MODE 3				SRSS
		Φ_1	$\frac{w}{g}\Phi_1$	$\frac{w}{g}\Phi_1^2$	a_1 (g)	Φ_2	$\frac{w}{g}\Phi_2$	$\frac{w}{g}\Phi_2^2$	a_2 (g)	Φ_3	$\frac{w}{g}\Phi_3$	$\frac{w}{g}\Phi_3^2$	a_3 (g)	a_x (g)
29	179.8	.0794	14.28	1.134	.104	.0747	13.43	1.003	-.106	.0684	12.30	.841	.094	.176
25	181.4	.0745	13.51	1.007	.098	.0411	7.46	.307	-.058	-.0040	-.73	.003	-.006	.114
21	181.4	.0666	12.08	.805	.087	-.0042	-.76	.003	.006	-.0644	-11.68	.752	-.089	.125
17	181.4	.0558	10.12	.565	.073	-.0471	-8.54	.402	.067	-.0630	-11.43	.720	-.087	.132
13	181.4	.0425	7.71	.328	.056	-.0718	-13.02	.935	.102	-.0023	-.42	.001	-.003	.116
9	181.4	.0279	5.06	.141	.037	-.0697	-12.64	.881	.099	.0604	10.96	.662	.083	.134
5	192.9	.0149	2.87	.043	.020	-.0467	-9.01	.421	.066	.0677	13.06	.884	.093	.116
GROUND	—	0	0	0	0	0	0	0	0	0	0	0	0	
Σ	1279.7		65.63	4.023			-23.08	3.952			12.06	3.863		
PF_{ROOF}	(Eq. 4-1)	$\frac{65.63}{4.023}(.0794) = 1.30$				$\frac{-23.08}{3.952}(.0747) = -.44$				$\frac{12.06}{3.863}(.0684) = .21$				$\Sigma = 1.07$
α	(Eq. 4-2)	$\frac{(65.63)^2}{(1279.7)(4.023)} = 0.837$				$\frac{(23.08)^2}{(1279.7)(3.952)} = 0.105$				$\frac{(12.06)^2}{(1279.7)(3.863)} = 0.029$				$\Sigma = 0.971$
T		3.00 sec				1.00 sec.				0.56 sec				
S_a		0.080 g				0.240 g				0.445 g				
a_{ROOF}	(Eq. 6-1)	$(1.30)(.080) = 0.104$ g				$(-.44)(.240) = -0.106$ g				$(.21)(.445) = 0.094$ g				.176
V	(Eq. 4-4)	$(.837)(.080)(41206) = 2759$ k				$(.105)(.24)(41206) = 1038$ k				$(.029)(.445)(41206) = 532$ k				2995 k
V/W		.067				.025				.013				.073

* Story 29 represents the roof, 29, 28 and one-half the 27th story. Stories 5 through 25 represent the indicated story plus 1½ stories above & below.

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Example E-1 7 of 7 Sample Modal Analyses

Figure E-1. Sample modal analysis—continued.

DESIGN EXAMPLE: E-2

BUILDING WITH A BOX SYSTEM:

Description of Structure. A 2-story hospital building with bearing walls in concrete, using a series of interior, vertical-load-carrying column and girder bents. The structural concept is illustrated in the Basic Design Manual, Design Example A-1.

Initial Trial Structure. The building in Design Example A-1 of the Basic Design Manual was designed for Z = 1.0 and I = 1.0 with a base shear coefficient V/W = ZIKCS = 0.186. In order to utilize the same structure in this example, the following conditions are assumed:

- Seismic Zone 3, Z = 3/4
- Hospital building, I = 1.5
- Box building, K = 1.33
- Soil factor, based on T_S = 2.5 sec
- Building period T < 0.3 sec
- CS = 0.133
- ZIKCS = 0.20

The base shear, V, for this example is 0.20W, which is close enough to that design base shear in the building in Design Example A-1 so that building will be used for the initial trial design.

Seismic Design Criteria. The building is to be designed in accordance with the dynamic analysis procedures of this manual. The following conditions apply:

- Building classification: Essential facility
- Ground motion spectra: ATC 3-06 spectra with $A_a = A_v = 0.30g$
- Soil profile coefficient: Type S_3

Design Procedure.

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Example E-2 1 of 23 Box System

Figure E-2. Building with a box system.

INTRODUCTION

THE SITE RESPONSE SPECTRA ARE DEVELOPED IN ACCORDANCE WITH THE PROCEDURE DESCRIBED IN CHAPTER 3. THE GOVERNING EQUATIONS AND SPECTRA FOR EQ-I AND EQ-II ARE SHOWN ON SHEETS 3 & 4, INCLUDING THE EFFECTS OF SITE SEVERITY, SOIL TYPE, AND STRUCTURAL DAMPING.

THE STRUCTURE OF EXAMPLE A-1 IN THE BASIC DESIGN MANUAL IS ASSUMED TO BE THE INITIAL TRIAL DESIGN AS DESCRIBED IN PARAGRAPH 5-3a OF THIS MANUAL. SINCE THE PERIOD OF VIBRATION IS SHORT, APPROXIMATELY .1 - .2 SECONDS, THE SPECTRAL ACCELERATION FOR EQ-I IS .28g. THIS Sa VALUE IS TWICE THE ZICS VALUE OF .14g WHICH WAS USED FOR THE TRIAL DESIGN, THEREFORE THE ANALYSIS FOR EQ-I WILL PROCEED WITHOUT MODIFYING THE STRUCTURE.

THE EXAMPLE STRUCTURE IS A BOX BUILDING WITH CONCRETE SHEAR WALLS IN BOTH THE TRANSVERSE AND LONGITUDINAL DIRECTIONS. THE METAL DECK ROOF SYSTEM FORMS A FLEXIBLE DIAPHRAGM WHILE THE METAL DECK WITH CONCRETE FILL FORMS A RIGID DIAPHRAGM AT THE SECOND FLOOR LEVEL.

IN ORDER TO PERFORM THE DYNAMIC MODAL ANALYSIS, BOTH THE ROOF AND 2ND FLOOR DIAPHRAGMS ARE ASSUMED TO BE RIGID. THE TRANSVERSE AND LONGITUDINAL ANALYSES ARE PERFORMED FOR THE BUILDING AS A WHOLE ASSUMING STRAIGHT LINE 1ST MODE SHAPES IN EACH DIRECTION. ONCE THE STORY SHEARS ARE DETERMINED, DISTRIBUTION TO INDIVIDUAL WALLS IS MADE CONSIDERING DIAPHRAGM FLEXIBILITY AND ANY ADDITIONAL CONTRIBUTION DUE TO TORSION.

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Example E-2 2 of 23 Box System

Figure E-2. Building with a box system—continued.

DETERMINATION OF SITE RESPONSE SPECTRA

SITE SEVERITY

FIGURES 3-40 TO 3-43 SHOW THE A_a AND A_v VALUES FOR AN ATC 3-06 SPECTRUM. SCALE FACTORS FOR EQ-I AND EQ-II ARE OBTAINED BY INTERPOLATION BETWEEN VALUES IN TABLE 3-4. VALUES USED IN THIS EXAMPLE ARE AS FOLLOWS:

	ATC 3-06	EQ-I	EQ-II
A_a	0.30g	0.14g	0.35g
A_v	0.30g	0.14g	0.35g

SOIL PROFILE COEFFICIENT, S_i

ASSUME SOIL PROFILE TYPE S_3

FROM TABLE 3-6, $S_i = 1.5$

DAMPING ADJUSTMENT FACTORS

DAMPING VALUES FROM TABLE 4-1, DAMPING FACTORS FROM TABLE 3-7

	DAMPING	DAMPING FACTOR
EQ-I	5%	1.0
EQ-II	10%	0.8

GOVERNING EQUATIONS (EQ. 3-27, 3-29)

EQ-I: $S_a = 1.22 A_v S_i / T = 1.22(.14)1.5/T = .2562/T \leq 2(A_a) = .28$

EQ-II: $S_a = .8(1.22)(.35)1.5/T = .5124/T \leq .8(2.0)(.35) = .56$.

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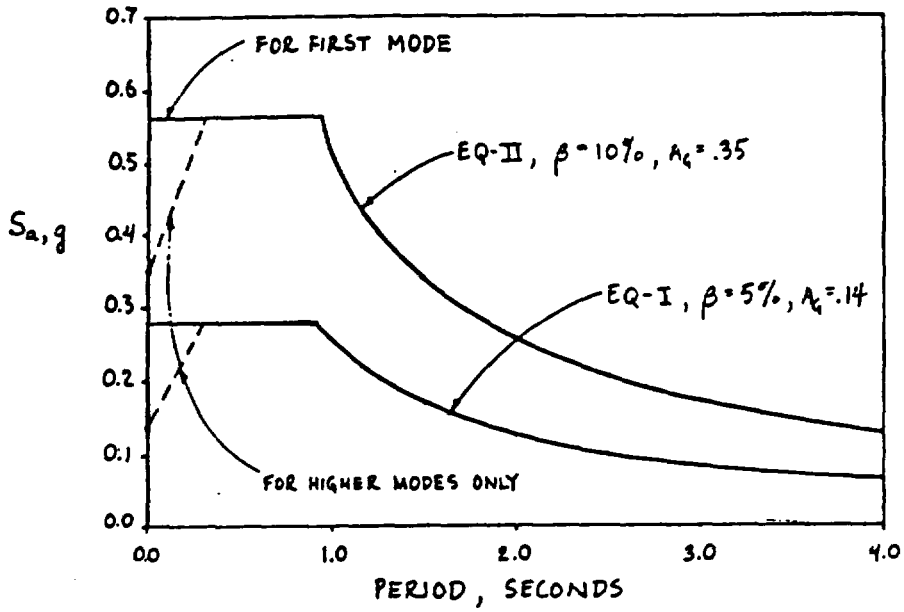
Example E-2 3 of 23 Box System

Figure E-2. Building with a box system—continued.

DESIGN RESPONSE SPECTRA FOR EQ-I AND EQ-II



	β		PERIOD							
			0.0	.3-.915	1.0	1.5	2.0	2.5	3.0	4.0
EQ-I	5%	S_a, g	.14	.28	.256	.171	.128	.102	.085	.064
EQ-II	10%	S_a, g	.35	.56	.512	.342	.256	.205	.171	.128

US Army Corps of Engineers

Example E-2 4 of 23 Box System

Figure E-2. Building with a box system—continued.

EQ - I

SPECTRAL ACCELERATION FOR FIRST MODE, S_{a1}

PERIOD $T = .05h/\sqrt{D}$

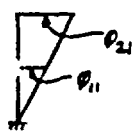
TRANSVERSE (N-S) $T = .05(23)/\sqrt{48} = .166$ SEC

LONGITUDINAL (E-W) $T = .05(22)/\sqrt{192} = .079$ SEC

$S_{a1} = .28g$ FROM EQ-I SPECTRUM FOR BOTH E-W AND N-S

MODE SHAPES, ϕ_{x1}

ASSUME A STRAIGHT LINE 1ST MODE SHAPE $\begin{Bmatrix} \phi_{21} \\ \phi_{11} \end{Bmatrix} = \begin{Bmatrix} 1.0 \\ .5 \end{Bmatrix}$



FIRST MODE BASE SHEAR, V_1 (SEE PAR. 4-3c FOR APPLICABLE EQUATIONS)

LEVEL	W_x, k	m_x, $\frac{k\text{-}sec^2}{ft.}$	ϕ_{x1}	$m_x\phi_{x1}$	$m_x\phi_{x1}^2$	PF_{x1}	a_{x1}, g	F_{x1}, k	V_{x1}, k
2	534	16.6	1.0	16.6	16.6	1.336	.374	200.0	200.0
1	1080	33.5	.5	16.8	8.4	.668	.187	202.0	402.0
Σ	1614	50.1		33.4	25.0			402.0	

$\alpha_1 = \frac{(33.4)^2}{(50.1)(25.0)} = .891$ 1ST MODE BASE SHEAR PARTICIPATION FACTOR (EQ. 4-2)

$C_{b1} = \alpha_1 S_{a1} = .891(.28) = .249$ 1ST MODE BASE SHEAR COEFFICIENT

$V_1 = \alpha_1 S_{a1} W = C_{b1} W = \underline{402\ k}$ 1ST MODE BASE SHEAR (EQ. 4-4)

US Army Corps of Engineers

Example E-2 5 of 23 Box System

Figure E-2. Building with a box system—continued.

DISTRIBUTION OF SEISMIC FORCES

SEISMIC FORCES FROM ROOF DIAPHRAGM TO WALLS BELOW

DIRECT SHEAR : THE ROOF SHEAR IS DISTRIBUTED BY TRIBUTARY AREA ($^{W}/_{\Sigma W}$) SINCE THE DIAPHRAGM IS FLEXIBLE.

TORSIONAL SHEAR: NO TORSION ASSUMED.

SEISMIC FORCES FROM 2ND FLOOR TO WALLS BELOW

DIRECT SHEAR : THE 2ND STORY SHEAR IS DISTRIBUTED ACCORDING TO THE RELATIVE RIGIDITIES OF THE WALLS BELOW ($^{R}/_{\Sigma R}$).

TORSIONAL SHEAR: THE TORSIONAL MOMENT, M_T, IS THE LARGER OF THE "CALCULATED" TORSION OR THE "ACCIDENTAL" TORSION DUE TO EITHER THE E-W OR N-S EARTHQUAKE. THE "ACCIDENTAL" TORSION IS COMPUTED USING THE ECCENTRICITIES WHICH RESULT BY MOVING THE CENTER OF MASS 5% OF THE MAXIMUM BUILDING DIMENSION TO EITHER SIDE OF ITS CALCULATED POSITION. THE TORSIONAL MOMENT IS RESISTED BY BOTH N-S AND E-W WALLS ACCORDING TO THEIR RIGIDITIES AND DISTANCE FROM THE CENTER OF RIGIDITY. THE TORSIONAL SHEARS ARE ADDED ALGEBRAICALLY TO THE DIRECT SHEARS.

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Example E-2 6 of 23 Box System

Figure E-2. Building with a box system—continued.

EQ-I : DISTRIBUTION OF SEISMIC FORCES FROM ROOF TO WALLS BELOW

TOTAL SHEAR BELOW ROOF $F_{x(NS)} = 200$ k
$F_{x(EW)} = 200$ k

TORSIONAL MOMENT $M_T = 0$

EQ-I	WALL	DIRECT SHEAR W, kips	DIRECT SHEAR R	DIRECT SHEAR DIRECT SHEAR	TORSIONAL SHEAR d	TORSIONAL SHEAR Rd^2	TORSIONAL SHEAR M_T	TORSIONAL SHEAR $\frac{Rd^2}{\ell Rd^2} \cdot \frac{M_T}{d}$	DIRECT SHEAR + TORSION
N-S	1	94	26.3	35.2					35.2
	3	169	38.1	63.3					63.3
	5	169	38.1	63.3					63.3
	7	102	55.5	38.2					38.2
		534		200.0					200.0
	A	267	35.6	0					
	C	267	35.6	0					
		534							
E-W	1	94	26.3	0					
	3	169	38.1	0					
	5	169	38.1	0					
	7	102	55.5	0					
		534							
	A	267	35.6	100.0					100.0
	C	267	35.6	100.0					100.0
		534		200.0					200.0

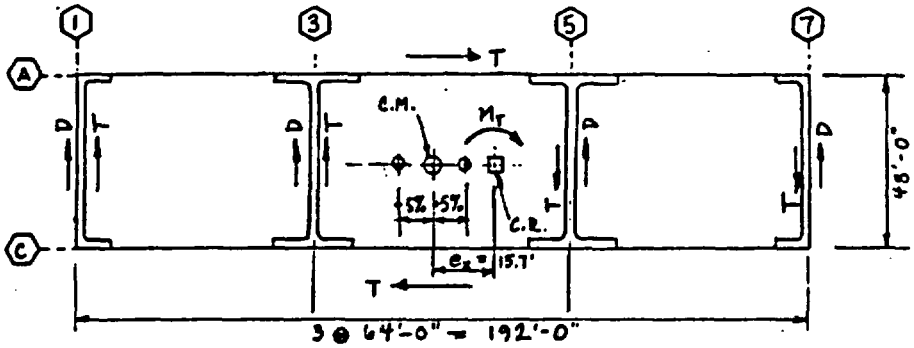
NO TORSION ASSUMED.

US Army Corps of Engineers

Example E-2 7 of 23 Box System

Figure E-2. Building with a box system—continued.

EQ. I: 2ND FLOOR PLAN SHOWING DIRECT SHEAR AND TORSIONAL SHEAR FORCES (SEE SHEET 10 FOR WALL ELEVATIONS)



NOTES: C.M. = CENTER OF MASS
C.R. = CENTER OF RIGIDITY
D = DIRECT SHEAR FORCE ($D = R/\Sigma R\ V_x$)
T = TORSIONAL SHEAR FORCE ($T = RA^2/\Sigma RA^2 \cdot M_T/d$)
M_T = TORSIONAL MOMENT

NORTH-SOUTH DIRECTION

FOR EACH WALL, USE THE "CALCULATED" OR "ACCIDENTAL" TORSIONAL MOMENT WHICH PRODUCES THE HIGHEST COMBINED SHEAR. IN THIS CASE, THE TORSIONAL SHEAR WILL ALWAYS BE ADDITIVE FOR WALLS 1 & 3 AND NEGATIVE FOR WALLS 5 & 7.

TORSIONAL MOMENT:

CALCULATED $M_T = V_x(e_x) = 402\,(15.7') = 6311$ ft-k

ACCIDENTAL $M_T = V_x(e_{max}) = V_x(e_x + .05 \times 192') = 402\,(25.3') = 10171$ ft-k

$M_T = V_x(e_{min}) = V_x(e_x - .05 \times 192') = 402\,(6.1') = 2452$ ft-k

EAST-WEST DIRECTION

USE THE "ACCIDENTAL" TORSIONAL MOMENT (W/EITHER SIGN ±).

$\pm M_T = V_x(.05 \times 192') = 402\,(9.6') = 3859$ ft-k

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Example E-2 8 of 23 Box System

Figure E-2. Building with a box system—continued.

EQ-I : DISTRIBUTION OF SEISMIC FORCES FROM 2ND FLOOR TO WALLS BELOW

TOTAL SHEAR BELOW 2ND FLR. DIAPHRAGM $F_{x(NS)} = 402k$
$F_{x(EW)} = 402k$

TORSIONAL MOMENT $M_{T(NS)}$ SEE SHEET 8
$M_{T(EW)} = 3859$ ft-k

EQ-I	WALL	DIRECT SHEAR			TORSIONAL SHEAR				DIRECT SHEAR + TORSION
		W, k	R	DIRECT SHEAR	d	Rd^2	GOVERNING M_T	$\frac{Rd^2}{\Sigma Rd^2} \cdot \frac{M_T}{d}$	
N-S	1		27.0	61.8	111.28	334347	10171	35.4	97.2
	3		44.9	102.8	47.7	102161	10171	25.2	128.0
	5		44.9	102.8	16.3	11929	2452	-2.1	100.7
	7		58.8	134.6	79.88	375192	2452	-13.3	121.3
			175.6	402.0					447.2
	A		35.6	0	23.58	19794	10171	9.9	9.9 *
	C		35.6	0	23.58	19794	10171	9.9	9.9 *
			71.2			863217			
E-W	1		27.0	0	111.28	334347	3859	13.4	13.4 *
	3		44.9	0	47.7	102161	"	9.6	9.6 *
	5		44.9	0	16.3	11929	"	3.3	3.3 *
	7		58.8	0	79.88	375192	"	21.0	21.0 *
			175.6						
	A		35.6	201.0	23.58	19794	"	3.8	204.8
	C		35.6	201.0	23.58	19794	"	3.8	204.8
			71.2	402.0		863217			409.6

* THESE VALUES ARE NOT CRITICAL, BUT ARE SHOWN HERE FOR CLARITY.

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Example E-2 9 of 23 Box System

Figure E-2. Building with a box system—continued.

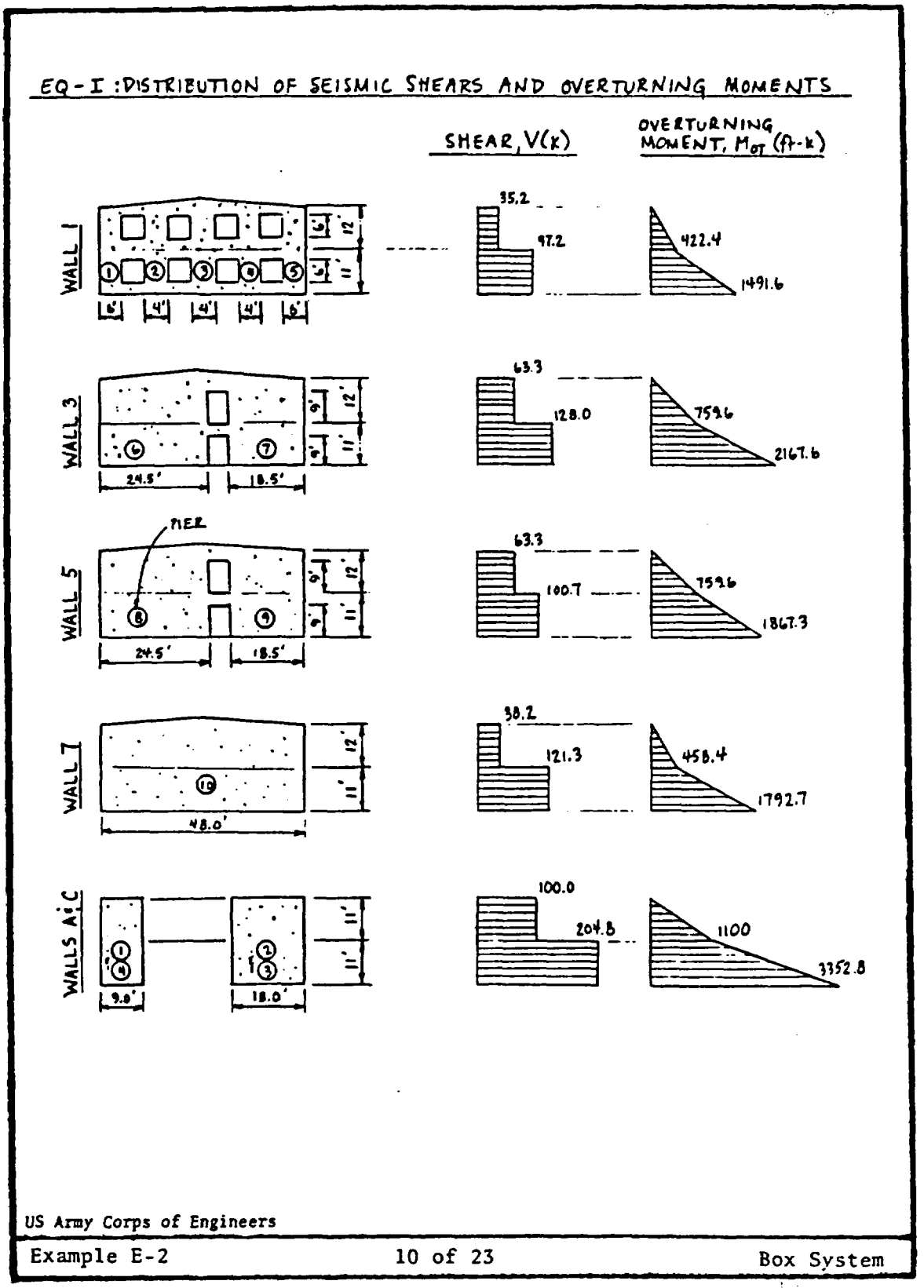

Figure E-2. Building with a box system—continued.

MOMENT CAPACITY - 1ST STORY WALLS, M_c

$M_c = M_u = \phi f_y A_s (d - \frac{a}{2})/12$ WHERE $\phi = .9$ FLEXURE
$f_y = 40$ ksi
$d = L - 2''$ COVER
$a = A_s f_y / .85 f'_c b$

WALL	TIER	L, in.	A_s*, in^2	a, in	$d-\frac{a}{2}$, in.	M_c, ft-k
1	1	72	.61	.72	69.6	127.4
	2	48	.61	.72	45.6	83.4
	3	48	.61	.72	45.6	83.4
	4	48	.61	.72	45.6	83.4
	5	72	.61	.72	69.6	127.4
	BASE	576	1.2	1.41	572.8	2062.
3,5	6	294	2.0	235	290.	1740.
	7	222	1.58	1.85	219.	1030.
7	8	576	2.0	2.35	572.	3431.
A, C	1	108	2.0	2.35	104.	624.
	2	216	3.0	2.53	212.	1905.
	3	216	3.0	2.53	212.	1905.
	4	108	2.0	2.35	104.	624.

*DATA FROM DESIGN MANUAL EXAMPLE A-1.

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Example E-2 11 of 23 Box System

Figure E-2. Building with a box system—continued.

EQ-I : CHECK DEMAND/CAPACITY RATIOS FOR PIERS IN 1ST STORY WALLS

SHEAR : $v_{DEMAND} \equiv v_D = V/\phi A_c$ $\quad v_{CAPACITY} \equiv v_C^{\dagger} = 2\sqrt{f'_c} + \rho f_y = 226$ psi

MOMENT: $M_{DEMAND} \equiv M_D = (\frac{R}{\Sigma R} M_{OT})$ OR $V(\frac{h}{2})$ FOR PIERS $\quad M_{CAPACITY} \equiv M_C = \phi f_y A_s (d - \frac{a}{2})/12$, $f_y = 40$ ksi

EQ-I	WALL	PIER	R* (k/in)	V (k)	A_c* (in^2)	v_D (psi)	M_D (k-ft)	A_s* (in^2)	M_C (k-ft)	v_D/v_C	M_D/M_C
N-S	1	1	11.1	30.2	720	49	91	.61	127	.22	.72
		2	4.5	12.3	480	30	37	.61	83	.13	.45
		3	4.5	12.3	480	30	37	.61	83	.13	.45
		4	4.5	12.3	480	30	37	.61	83	.13	.45
		5	11.1	30.2	720	49	91	.61	127	.22	.72
			35.7	97.2							
	1	BASE	27.0	97.2	5760	20	1492	1.2	2062	.09	.72
	3	6	27.0	77.0	2940	31	1303	2.0	1740	.14	.75
		7	17.9	51.0	2220	27	864	1.57	1030	.12	.84
			44.9	128.0			2167				
	5	8	27.0	60.6	2940	24	1123	2.0	1740	.11	.65
		9	17.9	40.1	2220	21	744	1.57	1030	.09	.72
			44.9	100.7			1867				
	7	10	58.8	121.3	5760	25	1793	2.0	3431	.11	.52
E-W	A,C	1	4.5	25.9	1080	28	424	2.0	624	.12	.68
		2	13.3	76.5	2160	42	1252	3.0	1905	.19	.66
		3	13.3	76.5	2160	42	1252	3.0	1905	.19	.66
		4	4.5	25.9	1080	28	424	2.0	624	.12	.68
			35.6	204.8			3352				

* DATA FROM DESIGN MANUAL EXAMPLE A-1.
† FROM EXAMPLE A-1 : $f'_c = 4$ ksi, $f_y = 40$ ksi, $\rho = .0025$

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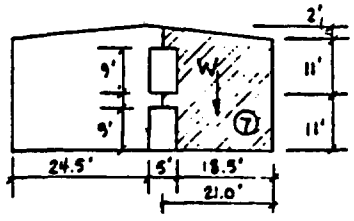
Example E-2 12 of 23 Box System

Figure E-2. Building with a box system—continued.

EQ-I : CHECK WALL 3, PIER 7 INCLUDING DEAD LOAD EFFECTS

THE PREVIOUS CALCULATIONS FOR DEMAND MOMENT WERE BASED ON EARTHQUAKE DEMAND ONLY, IGNORING THE DEAD LOADS. FOR WALLS WHICH ARE HIGHLY STRESSED DUE TO THE EARTHQUAKE LOADING, AN ADDITIONAL CHECK CAN BE MADE TO INCLUDE THE DEAD LOAD. FOR THE BEARING WALLS IN THIS EXAMPLE, THIS RESULTS IN A SUBSTANTIAL REDUCTION IN THE DEMAND TO CAPACITY RATIO. (THIS CALCULATION IS INCLUDED HERE FOR ILLUSTRATION ONLY SINCE THE WALLS IN THIS EXAMPLE ARE NOT OVERSTRESSED).



WALL 3

DEAD LOAD ON RIGHT PIER

ROOF	.0245 ksf × 32' × 21'	= 16.5
WALL	.125 ksf × 12' × 21	= 31.5
OPENING	−(.125) ksf × 2.5' × 9	= −2.8
		45.2 k

2ND FLOOR	.073 × 32' × 21'	= 49.06
WALL	.125 × 11' × 21'	= 28.88
OPENING	−(.125) × 2.5' × 9'	= −2.8
		75.1
		Σ 120.3

· ELASTIC CAPACITY (EQ. 4-6, 4-7)

$EC \geq 1.2D + 1.0L + 1.0E$ (COMPRESSION SIDE)

$EC \geq .8D + 1.0E$ (TENSION SIDE – MUST CHECK)

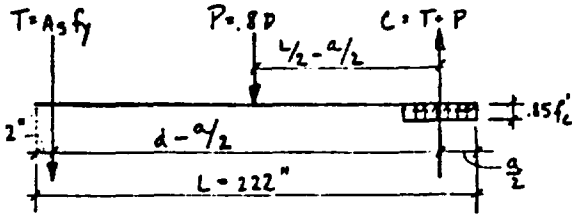
· PIER #7 (M_c FOR .8D)

$P = .8D = .8(120.3) = 96.2\ k$

$T = A_s f_y = 1.57(40) = 62.8\ k$

$C = T + P = 159.0\ k$

$a = \frac{C}{.85 f'_c b} = \frac{159.0}{.85(4)(10)} = 4.68''$



$M_c = \phi[T(d - a/2) + P(L/2 - a/2)] = .9[62.8(217.7'') + 96.2(108.7'')]/12 = 1810\ k\text{-}ft$

DEMAND/CAPACITY RATIO $M_D/M_c = 864/1810 = .48 < .84$

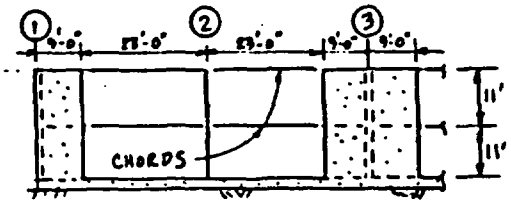
US Army Corps of Engineers

Example E-2 13 of 23 Box System

Figure E-2. Building with a box system—continued.

CHECK DEFLECTIONS & PERIOD FOR LONGITUDINAL WALLS A & C

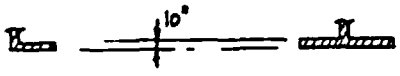
CONSIDER EACH PIER AS AN INDEPENDENT CANTILEVER AND COMPUTE THE DEFLECTIONS USING VIRTUAL WORK. ASSUME THE SHEAR IS DISTRIBUTED TO THE INDIVIDUAL PIERS IN ACCORDANCE WITH THEIR RELATIVE RIGIDITIES.



WALL A

$E = 3600 \cdot ksi = 518{,}400$

$G = .4E$



PIERS 1,4 PIERS 2 & 3

PROPERTIES

PIERS 1 & 4 : FOR CORNER PIER USE $I = 1.5(bd^3/12)$, $A_v = bd$

$I = 1.5(10/12)(9)^3/12 = 75.94 \text{ ft}^4$

$A_v = (10/12)(9) = 7.5 \text{ ft}^2$

PIERS 2 & 3 : RECTANGULAR PIER $I = (bd^3/12)$, $A_v = 5/6\,bd$

$I = (10/12)(18)^3/12 = 405 \text{ ft}^4$

$A_v = 5/6(10/12)(18) = 12.5 \text{ ft}^2$

RELATIVE RIGIDITIES

$\Delta = Ph^3/3EI + Ph/A_v G = h^3/3I + h/.4A_v$

PIER 1 : $\Delta = (22)^3/3(75.94) + 22/.4(7.5) = 54.07$ $R = \frac{1}{\Delta} = .0185$

PIER 2 : $\Delta = (22)^3/3(405) + 22/.4(12.5) = 13.16$ $R = \frac{1}{\Delta} = .0760$

$\Sigma R = .1889$

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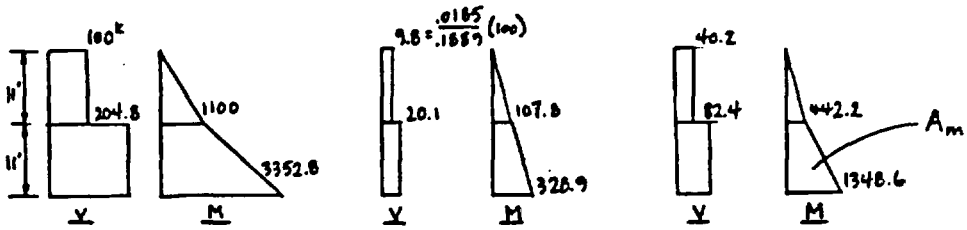
Example E-2 14 of 23 Box System

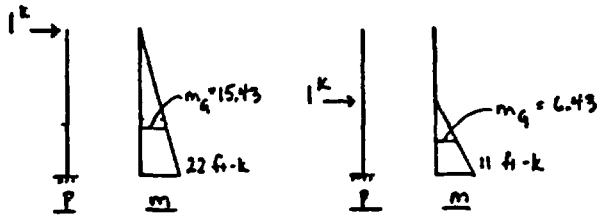
Figure E-2. Building with a box system—continued.

DEFLECTIONS (CONTINUED)

SHEAR DISTRIBUTION & OVERTURNING MOMENT



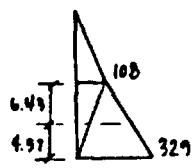
• VIRTUAL STRUCTURE FOR DISPLACEMENTS @ ROOF AND 2ND FLOOR



DEFLECTION OF PIERS 1 & 4

• BENDING DEFLECTION PIERS 1 & 4

$$EI\Delta = \int_0^L m M dx = A_m m_G$$



FOR Δ_{2ND}

$$A_m = \tfrac{1}{2}(11)(108) + \tfrac{1}{2}(11)(329) = 2404$$

$$C.G. = \frac{\left[\tfrac{2}{3}(11)(594) + \tfrac{1}{3}(1810)\right]}{2404} = 4.57'$$

$$m_G = 11\ ft\text{-}k\left(\tfrac{6.43}{11}\right) = 6.43$$

$$\Delta_{2ND} = A_m m_G / EI = 2404(6.43)12^2/518400(75.94)$$

$$= \underline{.0047''}$$

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Example E-2 15 of 23 Box System

Figure E-2. Building with a box system—continued.

DEFLECTIONS (CONTINUED)

FOR Δ_{ROOF}

$A_m = 2404 + \frac{1}{2}(11)(108) = 2998$

$C.G. = \frac{[4.57'(2404) + \frac{1}{3}(11)(594)]}{2998} = 6.57'$

$m_G = 22\ ft\text{-}k\ (15.43/22) = 15.43$

$\Delta_{ROOF} = A_m m_G / EI = 2998\ (15.43)(12^2)/518400\ (75.94)$
$= \underline{.0141''}$

• SHEAR DEFLECTION PIERS 1 & 4

$\Delta = 1.2Ph/AG = Ph/A_vG = Ph/A_v(.4E)$

$\Delta_{2ND} = 20.1\ (11)(12^2)/7.5(.4)(518400) = \underline{.0017''}$

$\Delta_{ROOF} = \Delta_{2ND} + 9.8(11)(12)/7.5(.4)(518400)$
$= .00171 + .000832 = \underline{.0025''}$

• TOTAL DEFLECTION PIERS 1 & 4

$\Delta_{2ND} = .0047 + .0017 = \underline{.0064''} = .39\ \Delta_{ROOF}$

$\Delta_{ROOF} = .0141 + .0025 = \underline{.0166''}$

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Example E-2	16 of 23	Box System

Figure E-2. Building with a box system—continued.

DEFLECTIONS (CONTINUED)

DEFLECTION OF PIERS 2 & 3

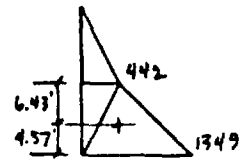
BENDING DEFLECTION $EI\Delta = \int mM\,dx = A_m m_G$

FOR Δ_{2ND}

$A_m = 11/2(442) + 11/2(1349) = 9851$

$C.G. = \dfrac{[2/3(11)(2431) + 11/3(7420)]}{9851} = 4.57'$

$m_G = 11\,(6.43/11) = 6.43$



$\Delta_{2ND} = A_m m_G / EI = 9851(6.43)(12)/518400(405)$
$= \underline{.0036''}$

FOR Δ_{ROOF}

$A_m = 9851 + 11/2(442) = 12282$

$C.G. = \dfrac{[4.57(9851) + 4/3(11)(2431)]}{12282} = 6.57'$

$m_G = 22\,(15.43/22) = 15.43$

$\Delta_{ROOF} = A_m m_G / EI = 12282(15.43)(12)/518400(405)$
$= \underline{.0108''}$

- SHEAR DEFLECTION PIERS 2 & 3

$\Delta_{2ND} = Ph/A_v(.4E) = 82.4(11)(12)/125(.4)(518400) = \underline{.0042''}$

$\Delta_{ROOF} = \Delta_{2ND} + 40.2(11)(12)/12.5(.4)(518400) = .0042 + .00205 = \underline{.0062''}$

- TOTAL DEFLECTION PIERS 2 & 3

$\Delta_{2ND} = .0036 + .0042 = \underline{.0078''} = .46\,\Delta_{ROOF}$

$\Delta_{ROOF} = .0108 + .0062 = \underline{.0170''}$

US Army Corps of Engineers

Example E-2 17 of 23 Box System

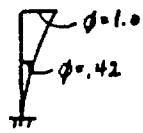
Figure E-2. Building with a box system—continued.

PERIOD OF LONGITUDINAL WALLS A & C

- PIERS 1,2,3 & 4 ARE CONNECTED BY CHORD MEMBERS AT THE ROOF AND 2ND FLOOR LEVELS WHICH CONSTRAIN THE PIERS TO DEFLECT TOGETHER. THUS, THE AVERAGE DEFLECTION OF THE 4 PIERS IS USED TO COMPUTE THE PERIOD OF THE WALL.

$$\Delta_{2ND} = \tfrac{2}{4}(.0064 + .0078) = \underline{.0071''} = .42\,\Delta_{ROOF}$$

$$\Delta_{ROOF} = \tfrac{2}{4}(.0166 + .0170) = \underline{.0168''}$$



- LONGITUDINAL PERIOD (E-W)

$$T = 2\pi\sqrt{\frac{\Sigma(W_x\,\delta_x^2)}{g\,\Sigma(F_x\,\delta_x)}}$$

$$= 2\pi\sqrt{\frac{534(.0168)^2 + 1080(.0071)^2}{386.4(200(.0168) + 209.6(.0071))}} = \underline{\underline{.066 \text{ SEC}}}$$

COMPARISON OF CALCULATED VALUES WITH INITIAL ASSUMPTIONS

LONGITUDINAL (E-W) DIRECTION

ASSUMED T = .079 SEC (SHEET 5)

CALCULATED T = .066 SEC

			ASSUMED		CALCULATED					
	W_x	m_x	ϕ_{x1}	F_{x1}	ϕ_{x1}	$m_x\phi_{x1}$	$m_x\phi_{x1}^2$	PF_{x1}	a_{x1}	F_{x1}
2	534	16.6	1.0	200.	1.0	16.6	16.6	1.364	.382	204
1	1080	33.5	.5	202.	.42	14.1	5.9	.573	.160	174
Σ	1614	50.1		402.		30.7	22.5			378

THE 2ND STORY SHEAR AND BASE SHEAR ARE LOWER, AND THE ROOF SHEAR IS ONLY 2% HIGHER THAN THE VALUES ASSUMED INITIALLY. THUS, THE INITIAL ASSUMPTIONS WERE ADEQUATE FOR THE EQ-I ANALYSIS.

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Example E-2 18 of 23 Box System

Figure E-2. Building with a box system—continued.

EQ-I COMMENTARY

- THE RATIOS OF THE ELASTIC DEMAND TO THE CAPACITY ARE LESS THAN ONE FOR ALL OF THE WALL ELEMENTS. THUS, THE SHEAR WALLS ARE ADEQUATE FOR AN EARTHQUAKE WITH THE CHARACTERISTICS OF EQ-I APPLIED IN EITHER THE TRANSVERSE OR THE LONGITUDINAL DIRECTION.

- ALTHOUGH THE EFFECTS OF TORSION INCREASE THE LOADS ON WALL 1 BY MORE THAN 50%, (i.e. $V_T = 35^k$ WHICH IS 36% OF THE TOTAL SHEAR FROM SHEET 9), THE RESULTING FORCES ARE SUBSTANTIALLY LESS THAN THE YIELD CAPACITY.

- CONSIDERATION OF THE DEAD LOAD IN THE CALCULATIONS FOR DEMAND MOMENT, M_v, WOULD REDUCE THE MOMENT DEMAND/CAPACITY RATIOS FOR THE BEARING WALLS IN THIS EXAMPLE, THUS ADDITIONAL SEISMIC CAPACITY IS AVAILABLE.

- THE DEFLECTION CALCULATIONS INDICATE THAT THE MODE SHAPES WHICH WERE ASSUMED INITIALLY ARE REASONABLY ACCURATE, THUS THE ASSUMED STRAIGHT LINE MODE SHAPES WILL ALSO BE USED FOR THE EQ-II ANALYSIS.

- THE CALCULATED E-W PERIOD OF 0.066 SECONDS IS CLOSE TO THE ASSUMED PERIOD OF 0.079 SECONDS. SINCE THE FIRST MODE SPECTRAL ACCELERATION IS CONSTANT FOR ALL PERIODS LESS THAN T = 0.9 SEC. (SEE SHEET 4), VARIATIONS IN THE CALCULATED PERIOD WILL NOT AFFECT THE EQ-I ANALYSIS.

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Example E-2	19 of 23	Box System

Figure E-2. Building with a box system—continued.

EQ - II

SPECTRAL ACCELERATION FOR FIRST MODE, S_{a_1}

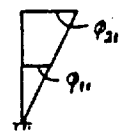
ASSUME PERIOD HAS LENGTHENED BY $\sqrt{1.25}$, WHERE 1.25 REPRESENTS THE INELASTIC SHEAR DEMAND RATIO FROM TABLE 4-2 FOR CONCRETE WALLS IN AN ESSENTIAL FACILITY.

TRANSVERSE (N-S) $T = \sqrt{1.25}(.17) = 0.19$ sec
LONGITUDINAL (E-W) $T = \sqrt{1.25}(.079) = 0.088$ sec

$S_{a_1} = .56g$ FROM EQ-II SPECTRUM FOR BOTH E-W & N-S

MODE SHAPES, ϕ_{x1}

ASSUME $\begin{Bmatrix} \phi_{21} \\ \phi_{11} \end{Bmatrix} = \begin{Bmatrix} 1.0 \\ .5 \end{Bmatrix}$



FIRST MODE BASE SHEAR, V_1

LEVEL	w_x, k	m_x, $\frac{k\,sec^2}{ft.}$	ϕ_{x1}	$m_x\phi_{x1}$	$m_x\phi_{x1}^2$	PF_{x1}	a_{x1}, g	F_{x1}, k	V_{x1}, k
2	534	16.6	1.0	16.6	16.6	1.336	.748	400.0	400.0
1	1080	33.5	.5	16.8	8.4	.668	.374	404.0	804.0
Σ	1614	50.1		33.4	25.0			804.0	

$$\alpha_1 = \frac{(33.4)^2}{(50.1)(25.0)} = .891$$

$C_{b1} = \alpha_1 S_{a1} = .891(.56) = .498$ MODAL BASE SHEAR COEFFICIENT

$V_1 = \alpha_1 S_{a1} W = C_{b1} W = \underline{804.0\ k}$ MODAL BASE SHEAR, EQ-II

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Example E-3 20 of 23 Box System

Figure E-2. Building with a box system—continued.

EQ-II

THE DEMANDS OF EQ-II ARE TWICE THOSE OF EQ-I ($S_{a(EQ-II)} = 0.56g$, $S_{a(EQ-I)} = 0.28g$). THEREFORE, ALL OF THE RATIOS OF v_D/v_c AND M_D/M_c ARE DOUBLED (SEE SHEET 12 OF 23). THE SHEAR INELASTIC DEMAND RATIOS ARE ALL LESS THAN 1.0 (FOR EXAMPLE - WALL 1, PIER 1: $v_D/v_c = 2 \times .22 = .44 < 1.0$). SOME MOMENT DEMAND RATIOS ARE GREATER THAN 1.0 (FOR EXAMPLE - WALL 3, PIER 7: $M_D/M_c = 2 \times .84 = 1.68 > 1.0$). HOWEVER WHEN DEAD LOAD EFFECTS ARE INCLUDED (SEE SHEET 13), THE INELASTIC DEMAND RATIOS ARE SIGNIFICANTLY REDUCED ($2 \times .48 = .84 < 1.0$). THUS, THE STRUCTURE REMAINS ESSENTIALLY ELASTIC FOR EQ-II FORCES.

NOTE: IF WALL 1 HAD INELASTIC DEMAND RATIOS SIGNIFICANTLY GREATER THAN 1.0 (i.e. $2 \times .72 = 1.44$) AND WALL 7 DID NOT (i.e. $2 \times .52 = 1.04$), WALL 1 WOULD YIELD AND THUS HAVE REDUCED STIFFNESS, THE C.R. OF THE BUILDING WOULD SHIFT TOWARDS WALL 7 (SEE SHEET 8) RESULTING IN A LARGER ECCENTRICITY, e_x. THIS TYPE OF CONDITION COULD LEAD TO TORSIONAL INSTABILITY. A CHECK FOR THIS CONDITION IS ILLUSTRATED ON THE FOLLOWING PAGE.

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Example E-3	21 of 23	Box System

Figure E-2. Building with a box system—continued.

TORSION CHECK (SEE PARA. 4-4c(5)(b), 5-5a(4)(d), 5-4i)

- ASSUME:
  - WALL 1 STIFFNESS REDUCED BY 1.5.
  - WALL 3 STIFFNESS REDUCED BY 1.5
  - ECCENTRICITY, e_x, INCREASES FROM 15.7' TO 29.0'

TORSIONAL MOMENT: (SEE SHEET 8)

CALCULATED $M_T = V_x e_x = 804\,(29')$ = 23316 k·ft

ACCIDENTAL $M_T = V_x e_{max} = 804\,(29'+9.6')$ = 31034 k·ft

$M_T = V_x e_{min} = 804\,(29'-9.6')$ = 15598 k·ft

	DIRECT SHEAR		TORSIONAL SHEAR				DIRECT + TORSION		
WALL	R	V_D	d	Rd^2	$V_T^{(1)}$	$V_T^{(2)}$	$V_D+V_T^{(1)}$	$V_D+V_T^{(2)}$	$2*EQ\text{-}I^{(3)}$
1	18.0	95	125	281250	75	100	170	195	194
3	30.0	159	61	111630	61	82	220	241	256
5	44.9	238	-3	404	-4	-3	234	235	201
7	58.8	312	-67	263953	-132	-88	180	224	243
	151.7	804					804		
A	35.6	0	23.6	19794					
C	35.6	0	23.6	19794					
				696825					

(1) BASED ON CALCULATED TORSIONAL MOMENT M_T = 23316.
(2) BASED ON GOVERNING ACCIDENTAL M_T = 31034 OR M_T = 15598.
(3) EQ-I VALUES FROM SHEET 9 (EQ-II = 2*EQ-I).

COMMENT. THE CASE SHOWN HERE ASSUMES THAT WALLS 1 & 3 HAVE REDUCED STIFFNESS DUE TO CRACKING AND THUS ATTRACT A SMALLER PROPORTION OF THE DIRECT SHEAR THAN IN THE ELASTIC ANALYSIS. THE SHIFT IN THE C.R. RESULTS IN A LONGER MOMENT ARM, d, THUS THEY ATTRACT MORE TORSIONAL SHEAR. A COMPARISON OF THE COMBINED SHEARS FOR THE ELASTIC CASE OR ASSUMED INELASTIC CASE (2 RIGHTHAND COLUMNS) SHOWS NO SUBSTANTIAL CHANGE, THEREFORE INELASTIC TORSION IS NOT CRITICAL.

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Example E-2 22 of 23 Box System

Figure E-2. Building with a box system—continued.

EQ-II : COMMENTARY

- THE RATIO OF THE SPECTRAL ACCELERATIONS FOR EQ-II TO EQ-I IS .56/.28 = 2.0 IN THIS EXAMPLE. THE DETAILS OF THE EQ-II ANALYSIS ARE NOT SHOWN HERE SINCE ALL OF EQ-I RESULTS ($V, M_T, M_{OT}, v_o/v_c, \Delta$, etc.) ARE INCREASED BY A FACTOR OF 2.0, AND THE CALCULATED PERIOD IS INCREASED BY $\sqrt{2.0}$. FOR EXAMPLE, PIER 7 OF WALL 3 WOULD HAVE A SHEAR OF $2 \times 51 = 102^k$, A MOMENT OF 1728 ft-k, AND INELASTIC DEMAND RATIOS OF .24 AND 1.68 FOR SHEAR AND BENDING, RESPECTIVELY.
- THE WALLS ON LINES 1 & 3 WILL YIELD BEFORE THE WALLS ON LINES 5 & 7, BUT THE EFFECTS OF INELASTIC TORSION WERE INVESTIGATED AND FOUND TO BE INSIGNIFICANT.
- TABLE 4-2 SHOWS THAT INELASTIC DEMAND RATIOS FOR CONCRETE WALLS IN AN ESSENTIAL FACILITY ARE NOT TO EXCEED 1.25 IN SHEAR OR 1.5 IN FLEXURE. THE RESULTS FOR EQ-II ARE SHOWN BELOW.

	LOCATION OF MAX.		INELASTIC DEMAND RATIOS	
	WALL	PIER	ACTUAL MAX.	ALLOWED MAX
SHEAR	1	1,5	0.44	1.25
FLEXURE	3	7	1.68*	2.0

*ACTUALLY LESS THAN 1.0 WHEN DEAD LOAD EFFECTS ARE INCLUDED.

THUS, THE SHEAR WALLS ARE ALSO ADEQUATE FOR EQ-II APPLIED IN EITHER THE TRANSVERSE OR LONGITUDINAL DIRECTION.

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Example E-2 23 of 23 Box System

Figure E-2. Building with a box system—continued.

DESIGN EXAMPLE: E-3

BUILDING WITH STEEL MOMENT-RESISTING SPACE FRAMES AND STEEL BRACED FRAMES:

Description of Structure. A 3-story hospital building with transverse ductile moment-resisting frames and longitudinal braced frames in structural steel, using nonstructural exterior curtain walls of flexible insulated metal panels. In addition, there are a series of interior vertical load-carrying column and girder bents. The structural concept is illustrated in the Basic Design Manual, design example A-3.

Initial Trial Structure. The building in design example A-3 of the Basic Design Manual was designed for a base shear (V = ZIKCSW) of 0.08W in the transverse direction and 0.14W in the longitudinal direction. In order to utilize the same structure in this example, the following conditions are assumed:

	Transverse	Longitudinal
Seismic Zone 3	Z = 3/4	Z = 3/4
Hospital building	I = 1.5	I = 1.5
Ductile frame/braced frame	K = 0.67	K = 1.0
Soil period	T_s = 1.0 sec	T_s = 1.0 sec
Building period	T = 0.69 sec	F = 0.3 sec
	CS = 0.116	CS = 0.140
	ZIKCS = 0.087	ZIKCS = 0.157

The above base shears (0.087W and 0.157W) are reasonably close to the base shears of the building in design example A-3 of the Basic Design Manual so that building will be used for the initial trial design.

Seismic Design Criteria. The building is to be designed in accordance with the dynamic analysis procedures of this manual. The following conditions apply:

Building classification: Essential facility
Ground motion spectra: ATC 3-06 spectra with $A_a = A_v = 0.30$
Soil profile coefficient: Type S_2

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Example E-3 1 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames.

Design Procedure. The site response spectra are developed in accordance with the procedure described in chapter 3. The governing equations and spectra for EQ-I and EQ-II, shown on sheets 3 and 4, include the effects of site severity, soil type, and structural damping. The structure of Basic Design Manual design example A-3 is assumed to be the initial trial design (para 5-3a). The EQ-I design spectrum is compared to the static base shear coefficients ZICS as follows:

	T, period (estimate)	$S_a(g)$	ZICS	Ratio $S_a \div$ ZICS
Transverse	0.69 sec	0.35	0.130	2.7
Longitudinal	0.3 sec	0.41	0.157	2.6

These ratios of S_a to ZICS are greater than 2. This is an indication that the structure may have to be modified for the higher force level. Because the ratio is less than 3, it has been decided to continue with the procedure without modifying the structure at this time.

The example building is a steel frame structure with lateral forces resisted by ductile frames in the transverse direction and braced frames in the longitudinal direction. The metal deck roof system forms a flexible diaphragm while the metal deck with concrete fill forms rigid diaphragms at the second- and third-floor levels. The procedure used to distribute the forces is discussed on sheet 5.

An outline of the procedures for the transverse direction and the longitudinal direction are given below:

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Example E-3 2 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

DETERMINATION OF SITE RESPONSE SPECTRA

SITE SEVERITY

SEE FIGURES 3-40 TO 3-43 AND TABLE 3-4

	ATC 3-06	EQ-I	EQ-II
A_a	0.30g	0.14g	0.35g
A_v	0.30g	0.14g	0.35g

SOIL PROFILE COEFFICIENT, S_i

ASSUME SOIL PROFILE TYPE S_2

FROM TABLE 3-6, $S_i = 1.2$

DAMPING ADJUSTMENT FACTORS

DAMPING FROM TABLE 4-1, DAMPING FACTORS FROM TABLE 3-7

	DAMPING	DAMPING FACTOR
EQ-I	3%	1.17
EQ-II	7%	0.90

GOVERNING EQUATIONS (EQ. 3-27, 3-28)

EQ-I: $S_a = 1.17(1.22)A_v S_i / T = 1.17(1.22)(.14)(1.2)/T$
$= .2398/T \leq 1.17(2.5)A_a = .41$

EQ-II: $S_a = .9(1.22)A_v S_i / T = .9(1.22)(.35)(1.2)/T$
$= .4612/T \leq .9(2.5)A_a = .79$

$EQ\text{-}II_{MIN} / EQ\text{-}I_{MAX} = 1.93$

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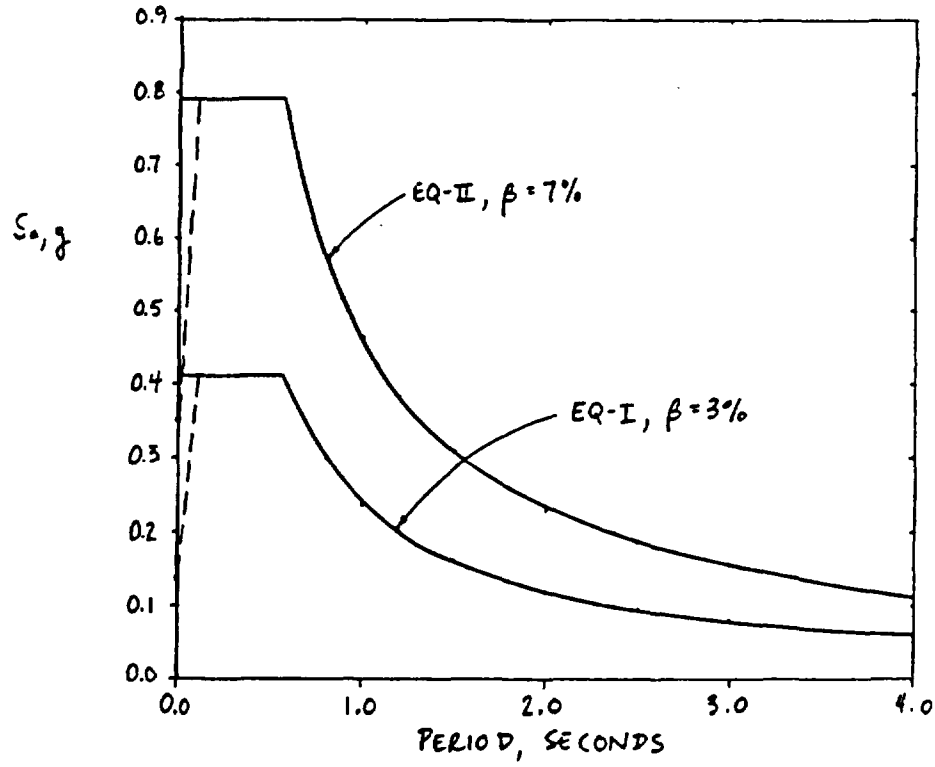
Example E-3 3 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

DESIGN RESPONSE SPECTRA FOR EQ-I AND EQ-II



EQ	β		PERIOD							
			0.0	.586	.80	1.0	1.5	2.0	3.0	4.0
EQ-I	3%	S_a, g	.14	.41	.300	.240	.160	.120	.080	.060
EQ-II	7%	S_a, g	.35	.79	.576	.461	.307	.231	.154	.115
		S_d^*, in	0	2.66	3.61	4.51	6.76	9.04	13.57	18.01

*SPECTRAL DISPLACEMENT $S_d = S_a (T/2\pi)^2 g$

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Example E-3 4 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

DISTRIBUTION OF FORCES FOR DYNAMIC ANALYSIS

THE DISTRIBUTION OF FORCES TO THE FRAMES WHICH WAS DEVELOPED FOR THE STATIC ANALYSES IS SHOWN ON PAGE 11 OF 34 OF EXAMPLE A-3 IN THE DESIGN MANUAL. IT IS ASSUMED THAT THE TRANSVERSE FRAMES ON LINES 1, 4 AND 7 ARE IDENTICAL, AS ARE THE TWO LONGITUDINAL FRAMES ON LINES A AND C. FORCES AT THE ROOF ARE DISTRIBUTED BY TRIBUTARY AREAS, BECAUSE OF A FLEXIBLE DIAPHRAGM, AND BY RELATIVE RIGIDITIES AT THE 2ND AND 3RD FLOORS. WHILE THERE IS NO "CALCULATED" TORSION IN THE BUILDING, AN "ACCIDENTAL" TORSIONAL SHEAR IS DISTRIBUTED TO FRAMES 1, 7, A, AND C.

FOR THE DYNAMIC ANALYSES, COMPUTER MODELS WERE DEVELOPED FOR FRAME 4, THE MOST HEAVILY LOADED OF THE THREE TRANSVERSE FRAMES, AND FRAME A, REPRESENTATIVE OF THE TWO LONGITUDINAL FRAMES. THE PROPERTIES OF THE FRAME 4 MODEL ARE SHOWN ON PAGE 17 OF 34 (EX. A-3). ONE HALF THE ROOF MASS AND ONE THIRD OF THE MASS AT EACH FLOOR ARE CARRIED BY FRAME 4 IN THE TRANSVERSE DIRECTION, CONSISTENT WITH THE DISTRIBUTION DISCUSSED ABOVE. FOR THE LONGITUDINAL ANALYSES, ONE HALF THE BUILDING MASS IS TAKEN BY FRAME A AT EACH LEVEL. THE SHEARS RESULTING FROM THE EQ-I AND EQ-II MODAL ANALYSES WILL BE INCREASED BY 18% ($.59F_L/.50F_L$) IN ORDER TO ACCOUNT FOR THE TORSIONAL SHEAR AT THE 2ND AND 3RD FLOORS.

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Example E-3 5 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

TRANSVERSE (N-S) DIRECTION : FRAME 4 - DMRSF

MODES SHAPES (ϕ_{xm}) AND PERIODS (T_m) FROM COMPUTER ANALYSIS OF FRAME 4, MASS CALCULATED FROM W/g.

LEVEL	MASS ($\frac{k\text{-}sec^2}{ft}$)	MODE 1			MODE 2			MODE 3		
		ϕ_{x1}	$m_x\phi_{x1}$	$m_x\phi_{x1}^2$	ϕ_{x2}	$m_x\phi_{x2}$	$m_x\phi_{x2}^2$	ϕ_{x3}	$m_x\phi_{x3}$	$m_x\phi_{x3}^2$
R	5.81	.3320	1.929	.640	.2384	1.385	.330	.0713	.4143	.030
3	7.32	.2044	1.496	.306	-.2201	-1.611	.355	-.2154	-1.577	.340
2	7.32	.0860	.630	.054	-.2075	-1.519	.315	.2936	2.149	.631
Σ	20.45		4.055	1.000**		-1.745	1.000		.9863	1.001
PF^*_{Rm} (Eq. 4-1)		$\frac{\Sigma m\phi}{\Sigma m\phi^2}\cdot\phi_{R1}$ =		1.346		-.416			.070	
PF_{3m}				.829		.384			-.212	
PF_{2m}				.349		.362			.289	
α_m (Eq. 4-2)		$\frac{(\Sigma m\phi)^2}{\Sigma m(\Sigma m\phi^2)}$ =		.8040		.149			.048	
T_m, sec.				.964		.356			.182	

*AS A CHECK, NOTE THAT $\sum_{m=1}^{3} PF_{Rm} = 1.0$ AND $\sum_{m=1}^{3} \alpha_m = 1.0$.

**THE COMPUTER PROGRAM NORMALIZES THE RESULTS SO THAT $\Sigma m\phi^2 = 1.0$.

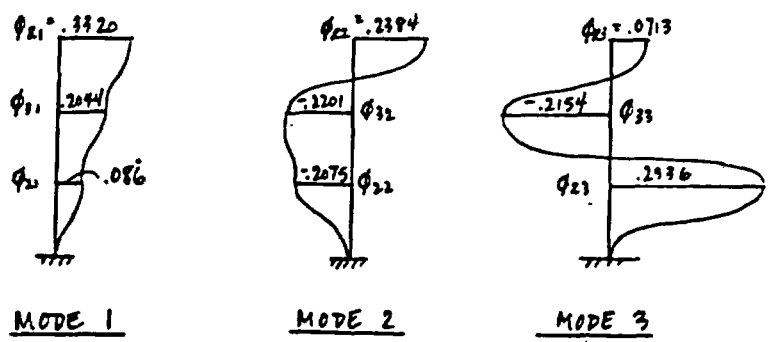

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Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

TRANSVERSE (N-S) DIRECTION : SPECTRAL ACCELERATIONS, S_{am}
MODAL BASE SHEARS, V_m

EQ		MODE 1	MODE 2	MODE 3
EQ-I (β=3%)	T_m, sec (SHEET 6)	.964	.356	.182
	S_{am}, g (SHEET 4)	.251	.41	.41
	$C_{bm}=\alpha_m S_{am}$	.202	.061	.020
	$V_m=C_{bm}W=C_{bm}(\Sigma mg)$ (EQ. 4-4)	132.7 k	40.2 k	13.0 k
EQ-II (β=7%)	$T_m\sqrt{1.25}$*, sec	1.078	.398	.204
	S_{am}, g (SHEET 4)	.428	.79	.79
	$C_{bm}=\alpha_m S_{am}$	.344	.118	.038
	$V_m=C_{bm}W$	226.5 k	77.5 k	25.0 k

* NOTE : FOR THE EQ-II ANALYSIS, AS A ROUGH APPROXIMATION, ASSUME THE PERIOD HAS LENGTHENED BY $\sqrt{X}$, WHERE X REPRESENTS THE INELASTIC DEMAND RATIO FOR THE CRITICAL ELEMENTS IN THE FRAME AS SHOWN IN TABLE 4-2. IN THIS EXAMPLE, X=1.25 FOR COLUMNS IN A STEEL DMRSF IN A CRITICAL AND ESSENTIAL FACILITY. THE COMPUTER MODEL USED FOR THE EQ-II ANALYSIS HAS A REDUCED ELASTIC MODULUS IN ORDER TO OBTAIN THE LONGER PERIOD ($E_{EQ-II}=E_{EQ-I}/X$).

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Example E-3 7 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

MODAL ANALYSIS: TRANSVERSE (N-S) DIRECTION - FRAME 4

		LEVEL	PF_{xm}	$\frac{m_x \phi_{xm}}{\Sigma m_x \phi_{xm}}$	F_{xm} (k)	V_{xm} (k)	ΔOTM_{xm} (ft-k)	OTM_{xm} (ft-k)	$a_{xm} = \frac{F_{xm}}{W_x}$	δ_{xm} (in)	Δ_{xm} (in)
MODE 1	EQ-I	R	1.346	.476	63.2	63.2	772	0	.337	3.065	1.182
		3	.829	.369	48.9	112.1	1233	772	.208	1.892	1.101
		2	.349	.155	20.6	132.7	1416	2005	.087	.791	.791
				1.000				3421			
	EQ-II	R			107.8	107.8	1316	0	.576	6.551	2.525
		3			83.6	191.4	2105	1316	.354	4.026	2.331
		2			35.1	226.5	2417	3421	.149	1.695	1.695
								5838			
MODE 2	EQ-I	R	-.416	-.793	-31.9	-31.9	-389	0	-.171	-.212	.407
		3	.384	.923	37.1	5.2	57	-389	.157	.195	.011
		2	.362	.870	35.0	40.2	429	-332	.148	.184	.184
				1.000				97			
	EQ-II	R			-61.4	-61.4	-750	0	-.329	-.612	1.176
		3			71.5	10.1	111	-751	.303	.564	.032
		2			67.4	77.5	827	-639	.286	.532	.532
								188			
MODE 3	EQ-I	R	.070	.420	5.5	5.5	67	0	.029	.0094	.037
		3	-.212	-1.599	-20.8	-15.3	-168	67	-.087	-.028	.066
		2	.289	2.179	28.3	13.0	139	-101	.118	.038	.038
				1.000				38			
	EQ-II	R			10.5	10.5	128	0	.055	.027	.108
		3			-40.0	-29.5	-324	128	-.167	-.081	.191
		2			54.5	25.0	267	-196	.228	.110	.110
								70			
SRSS	EQ-I	R			71.0	71.0	867	0	.379	3.072	1.251
		3			64.8	113.3	1246	867	.275	1.893	1.094
		2			49.5	139.3	1486	2035	.208	.812	.813
								3423			
	EQ-II	R			124.5	124.5	1520	0	.666	6.580	2.788
		3			117.1	193.9	2133	1520	.495	4.066	2.339
		2			93.5	240.7	2568	3653	.395	1.780	1.780
								6221			

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Example E-3 8 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

MODAL ANALYSIS - INFLUENCE OF HIGHER MODES

HIGHER MODES OF RESPONSE BECOME INCREASINGLY IMPORTANT AS A BUILDING GETS TALLER OR MORE IRREGULAR. FOR THIS REGULAR 3-STORY STRUCTURE, THE FIRST MODE DOMINATES THE LATERAL RESPONSE. A COMPARISON OF THE MODAL STORY SHEARS AND THE SRSS STORY SHEARS IS SHOWN BELOW. FOR EXAMPLE, IF ONLY THE 1ST MODE SHEARS HAD BEEN USED FOR ANALYSIS, THIS REPRESENTS 89% OF THE SRSS SHEAR AT THE ROOF, 99% AT THE 3RD FLOOR AND 95% AT THE 2ND FLOOR. WHILE THE 2ND MODE SHEAR AT THE ROOF IS 50% OF THE 1ST MODE SHEAR, WHEN COMBINED ON AN SRSS BASIS THE 1ST MODE ACCOUNTS FOR 79% OF THE SRSS RESPONSE WITH 20% FOR THE 2ND MODE AND 0.6% FOR THE 3RD MODE. THESE PERCENTAGES ARE 91%, 8% AND 1% AT THE BASE.

STORY SHEARS - EQ-I

LEVEL	V_{SRSS}	MODE 1			MODE 2		MODE 3	
		V_1	V_1/V_{SRSS}	$(V_1/V_{SRSS})^2$	V_2	$(V_2/V_{SRSS})^2$	V_3	$(V_3/V_{SRSS})^2$
R	71.0	63.2	.89	.79	-31.9	.202	5.5	.006
3	113.3	112.1	.989	.98	5.2	.002	-15.3	.018
2	139.3	132.7	.953	.91	40.2	.083	13.0	.009

THE EFFECTIVE MODAL WEIGHT FACTOR, α_m, ALSO SHOWS THE RELATIVE IMPORTANCE OF EACH MODE. IN THIS EXAMPLE, (α_1=.804, α_2=.149, α_3=.048), 80.4% OF THE BUILDING MASS PARTICIPATES IN THE 1ST MODE, 14.9% IN THE 2ND MODE AND 4.8% IN THE 3RD MODE.

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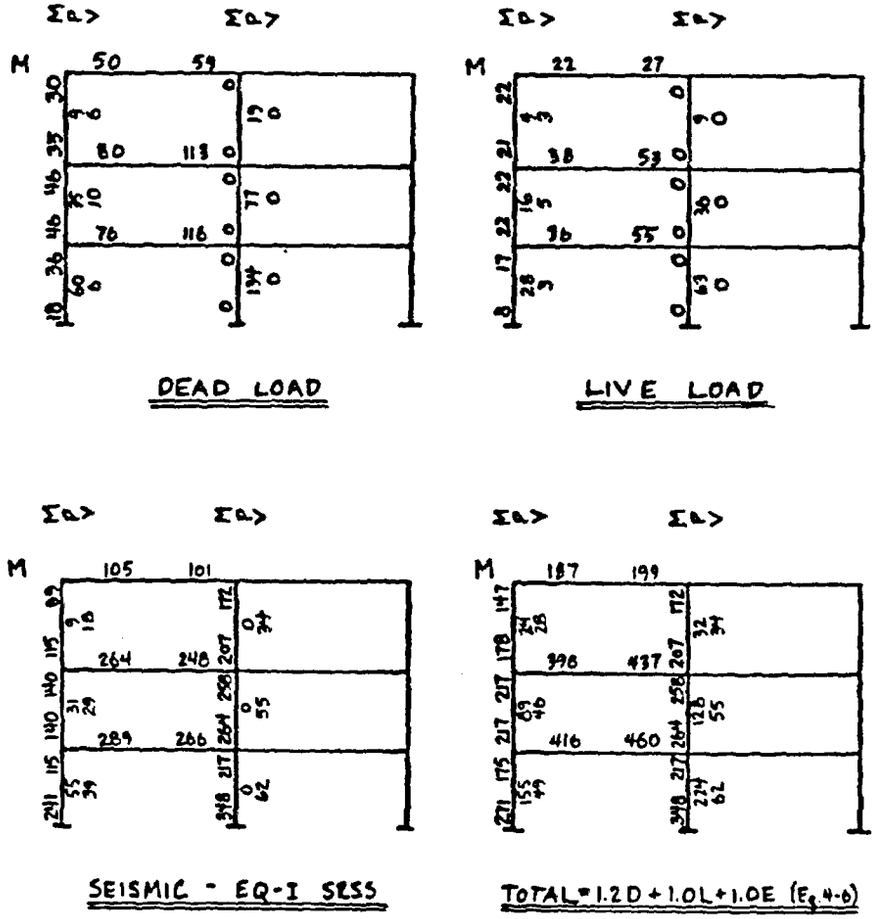
Example E-3 9 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

EQ-I ELEMENT FORCES : TRANSVERSE (N-S) DIRECTION - FRAME 4

- DL & LL RESULTS FROM DESIGN MANUAL EXAMPLE A-3, SHEET 18 OF 34.
- SEISMIC RESULTS FROM COMPUTER ANALYSIS.
- ALL END MOMENTS AND SHEARS GIVEN AT FACE OF SUPPORT.
- UNITS ARE K, K-ft.



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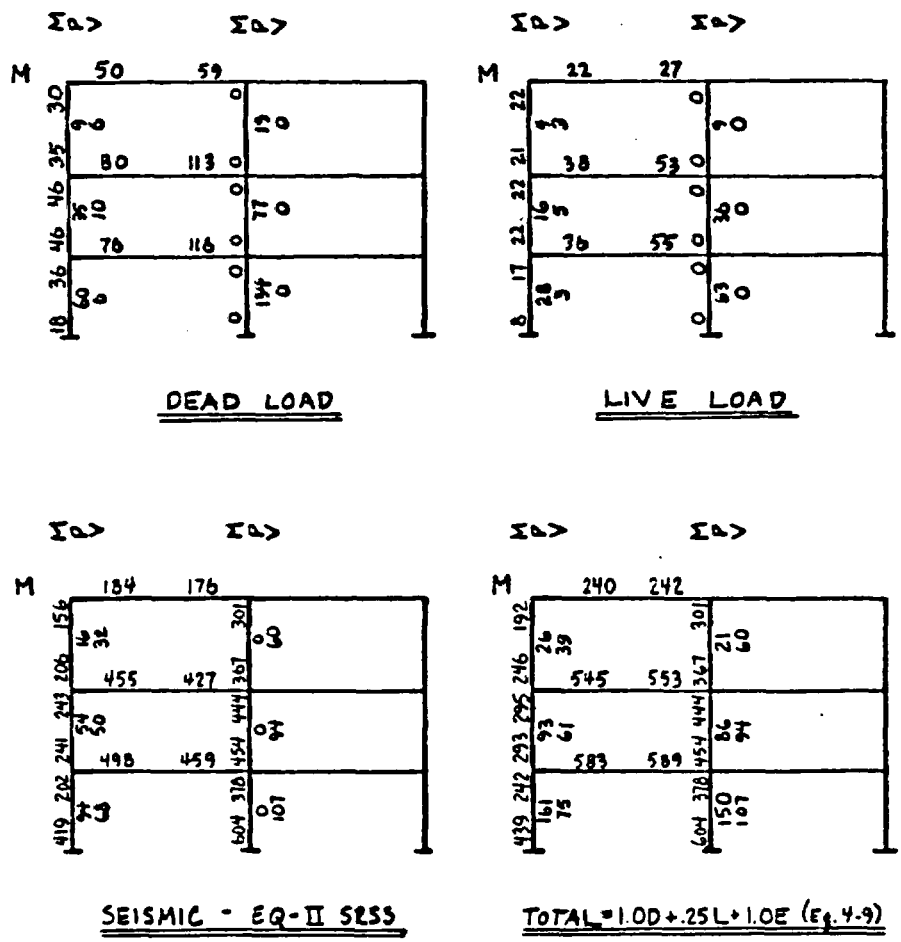
Example E-3 10 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

EQ-II ELEMENT FORCES : TRANSVERSE (N-S) DIRECTION - FRAME 4

- DL & LL RESULTS FROM DESIGN MANUAL EXAMPLE A-3, SHEET 18 OF 34.
- SEISMIC RESULTS FROM COMPUTER ANALYSIS.
- ALL END MOMENTS AND SHEARS GIVEN AT FACE OF SUPPORT.
- UNITS ARE K, K-ft.



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Example E-3 11 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

CHECK ELEMENT STRESSES

FOR EQ-I USE 1.7 * ALLOWABLE STRESSES (SEE AISC, 8TH EDITION, SECTION 1.5) OR USE STRENGTH DESIGN CRITERIA. COMPARE DEMAND FORCES TO CAPACITY FORCES AND REVIEW FOR ELASTIC, OR NEARLY ELASTIC BEHAVIOR (SEE para. 4-3c(1)). FOR THE DMRSF IN THE TRANSVERSE DIRECTION, 20% OF THE BEAMS AND 10% OF THE COLUMNS AT ANY STORY ARE ALLOWED TO EXCEED THE FLEXURAL STRENGTH REQUIREMENTS BY UP TO 25%. FOR THE BRACED FRAMES IN THE LONGITUDINAL DIRECTION (K=1.0), 20% OF THE BEAMS AND 10% OF THE COLUMNS AT ANY STORY ARE ALLOWED TO EXCEED THE FLEXURAL STRENGTH REQUIREMENTS BY UP TO 10%. NO OVERSTRESS IS ALLOWED FOR THE K-BRACES.

FOR EQ-II COMPARE DEMAND FORCES TO THE PLASTIC MEMBER CAPACITIES IN ORDER TO COMPUTE THE INELASTIC DEMAND RATIOS. SEE FIGURE 4-1 FOR STEEL BEAMS AND FIGURE 4-2 FOR STEEL COLUMNS. THE ALLOWABLE DUCTILITIES OR INELASTIC DEMAND RATIOS ARE SHOWN IN TABLE 4-2. FOR THE TRANSVERSE DMRSF, THE ALLOWABLE RATIOS ARE 2.0 FOR BEAMS AND 1.25 FOR COLUMNS (EXCEPT P/P_{cr} MUST BE LESS THAN 1.0). FOR THE BRACED FRAMES, THE ALLOWABLE RATIOS ARE 1.5 FOR BEAMS, 1.25 FOR COLUMNS, AND 1.0 FOR K-BRACES.

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Example E-3 12 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

ELEMENT STRESSES - FRAME 4 (DMRSF)

BEAM ELEMENTS : EQ-I $EC \geq 1.2D + 1.0L + 1.0E$ (EQ. 4-6)
EQ-II $UC \geq 1.0D + .25L + 1.0E$ (EQ. 4-9)

LEVEL	SIZE.	Z_x (in^3)	EQ-I M_D (k-ft)	EQ-I M_C* (k-ft)	EQ-I M_D/M_C	EQ-II M_D (k-ft)	EQ-II M_C* (k-ft)	EQ-II M_D/M_C	IDR**
ROOF	W14×30	47.3	199	142	1.40	242	142	1.70	2.0
3	W18×55	112	437	336	1.30	553	336	1.65	2.0
2	W18×60	123	460	369	1.25	589	369	1.60	2.0

*USE $M_C = M_{px} = Z_x F_y$, $F_y = 36$ ksi
** INELASTIC DEMAND RATIO FROM TABLE 4-2

COMMENT
- FOR EQ-I, THE RATIO OF MOMENT DEMAND TO MOMENT CAPACITY IS LIMITED TO 1.0 FOR MOST ELEMENTS BUT IS ALLOWED TO REACH UP TO 1.25 FOR A LIMITED NUMBER OF ELEMENTS IN ACCORDANCE WITH THE "NEARLY ELASTIC" CRITERIA (SEE PARA. 4-3e(1)(a)). THESE LIMITS HAVE BEEN EXCEEDED FOR EQ-I.

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Example E-3 13 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

ELEMENT STRESSES - FRAME 4 (CONTINUED)

STEEL COLUMNS 1ST STORY @ BASE (SEE DESIGN MANUAL EXAMPLE A-3 SHEET 21 OF 34)

EQ-I $EC \geq 1.2D + 1.0L + 1.0E$

SIZE	A (in^2)	S_x (in^3)	EQ - I								
			P_D (k)	M_D (k-ft)	f_a (ksi)	f_{bx} (ksi)	F_a (ksi)	F_{bx} (ksi)	F'_{ex} (ksi)	** EQ1.6-1a	** EQ.1.6-1b
W14x48	14.1	70.3	155	271	11.0	46.3	17.2	24	361	0.84	1.43
W14x61	17.9	92.2	224	348	12.5	45.3	18.4	24	377	0.85	1.45

** MODIFIED UNIAXIAL INTERACTION EQUATIONS USING $1.7F_{allow}$

EQ1.6-1a $\dfrac{f_a}{1.7F_a} + \dfrac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{1.7F'_{ex}}\right)1.7F_{bx}} \leq 1.0$, EQ1.6-1b $\dfrac{f_a}{1.7(.6F_y)} + \dfrac{f_{bx}}{1.7F_{bx}} \leq 1.0$

EQ-II $UC \geq 1.0D + .25L + 1.0E$

SIZE	A (in^2)	Z_x (in^3)	EQ- II								
			P_D (k)	M_D (k-ft)	M_D (k-in)	$P_y = AF_y$ (k)	$P_c = 1.7AF_a$ (k)	$M_{px} = Z_x F_y$ (k-in)	M_{pcx} (k-in)	$\frac{M_x}{M_{pcx}}$ †	IDR (TABLE 4-2)
W14x48	14.1	78.4	161	439	5268	507.6	412.3	2822	2333	2.26	1.25
W14x61	17.9	102	150	604	7248	644.4	559.9	3672	3324	2.18	1.25

† SEE FIGURE 4-2, $M_x / M_{cpx} < \mu$ (μ = IDR)

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Example E-3 14 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

CHECK INTERSTORY DRIFT

ALLOWABLE DRIFT FOR ESSENTIAL FACILITIES:
.005 * STORY HEIGHT FOR EQ-I (par. 4-3(c)(7)(a))
.010 * STORY HEIGHT FOR EQ-II (par. 4-4(c)(2)(a))

TRANSVERSE (N-S) DIRECTION - FRAME 4

LEVEL	STORY HT. h, in	EQ - I		EQ - II	
		Δ_{SESS}, in*	.005h	Δ_{SESS}, in*	.010h
ROOF	147	1.251	.733	2.788	1.465
3	132	1.094	.660	2.339	1.320
2	128	.813	.640	1.780	1.280

* Δ_{SESS} VALUES FROM SHEET 8.

• THE ALLOWABLE DRIFT LIMITS ARE EXCEEDED AT EVERY LEVEL OF FRAME 4 FOR BOTH EQ-I AND EQ-II.

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Example E-3 15 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

FRAME 4 (DMRSF) - COMMENTARY

EQ-I. THE ELASTIC ANALYSIS OF FRAME 4 FOR EQ-I SHOWS THAT THE FIRST FLOOR COLUMNS AND ALL OF THE BEAMS ARE OVERSTRESSED, AND THE ALLOWABLE DRIFT LIMITS HAVE BEEN EXCEEDED. THE FRAME HAS EXCEEDED THE "NEARLY ELASTIC" CRITERIA SINCE THE OVERSTRESS RATIOS FOR BOTH BEAMS AND COLUMNS ARE GREATER THAN THE 1.25 ALLOWED FOR DUCTILE FRAMING SYSTEMS.

FRAME 4 INCLUDES 1/3 OF THE SEISMIC RESISTING ELEMENTS IN THE TRANSVERSE DIRECTION, AND WAS INITIALLY SELECTED FOR ANALYSIS BECAUSE IT CARRIES MORE LOAD THAN EITHER OF THE END FRAMES. A SIMILAR ANALYSIS FOR FRAMES 1 & 7 MIGHT RESULT IN LOWER STRESS RATIOS FOR THESE FRAMES BUT THE BUILDING AS A WHOLE STILL WOULD NOT MEET THE "NEARLY ELASTIC" CRITERIA.

EQ-II - METHOD 1 THE EQ-II ANALYSIS FOLLOWED THE ELASTIC PROCEDURE DESCRIBED AS METHOD 1 IN PARAGRAPH 4-4c. THE INELASTIC DEMAND RATIOS FOR THE FIRST FLOOR COLUMNS ARE GREATER THAN THE 1.25 WHICH IS ALLOWED, AND THE DRIFT LIMITS HAVE BEEN EXCEEDED.

EQ-II - METHOD 2 WHILE FRAME 4 DOES NOT HAVE SUFFICIENT CAPACITY TO RESIST EITHER EQ-I OR EQ-II, A FURTHER CHECK WAS PERFORMED TO SHOW AN EXAMPLE OF METHOD 2 DESCRIBED IN PARAGRAPH 4-4d. THIS IS SHOWN ON THE FOLLOWING PAGES.

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Example E-3 16 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

METHOD 2: CAPACITY SPECTRUM METHOD FOR TRANSVERSE DIRECTION RESPONSE TO EQ-II
REFER TO PARA 4-4d AND PARA 5-5b.

DETERMINE ELASTIC CAPACITY RATIO TO EQ-I.
USE EQ-II LOAD FACTORS. (PARA 5-5b(3)(a)-(d)

BEAM ELEMENTS: REFER TO SHEETS 10 AND 13

LEVEL	SIZE	EC	D	.25L	NET*	E_I	ECR
ROOF	W14×30	142	59	7	76	101	0.75
3RD	W18×55	336	113	13	210	248	0.85
2ND	W18×60	369	116	14	239	266	0.90

* NET EARTHQUAKE CAPACITY = EC − D − .25L (eq 5-7)
E_I = EQ-I DEMANDS
ECR = ELASTIC CAPACITY RATIO = NET*/E_I

INDICATES FIRST YIELD AT 0.75 EQ-I – BUT CHECK THE COLUMNS

COLUMN ELEMENTS: REFER TO SHEETS 10 AND 14

LEVEL	SIZE	0.75 EQ-I P_D	0.75 EQ-I M_D	f_a	f_b	EQ 1.6-1b SHT 14	
BASE	W14×48	108	201	7.7	34.3	1.13	YIELD AT BASE PRIOR TO BMS.
BASE	W14×61	150	261	8.4	34.0	1.06	
BASE△	W14×48	26	162	1.2	27.6	0.77	

P_D AND M_D = D + 0.25L + 0.75(EQ-I)
△ SEISMIC FORCES OPPOSITE DIRECTION TO GRAVITY.

FIRST YIELD: $\frac{0.75}{1.13}$ × EQ-I = 0.66 TIMES EQ-I

(REFER TO SHEET 7)	MODE 1	MODE 2	MODE 3	SRSS
BASE SHEAR COEF, C_B, EQ-I	0.202	0.061	0.020	0.212
YIELD AT 0.66 × EQ-I	0.133	0.040	0.013	0.140

CAPACITY CURVE DATA: YIELD SRSS, C_B = 0.14
1st MODE, C_B = 0.13
USE 1ST MODE VALUES TO PLOT CURVE. SEE SHEET 18, 19, 20

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Example E-3 17 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

It has been determined that the seismic base shear coefficient (in terms of first mode values) could reach a value of 0.13 before any yielding would occur in the structural frame. For seismic forces applied towards the north (towards the right on sheet 19), the base of the north (right) column and the center column will yield in flexure (the column bases were assumed fixed). The south (left) column does not yield because both the dead and live load stresses are counterbalancing some of the lateral load stresses. At a base shear coefficient of 0.13, the spectral acceleration is 0.161g, the spectral displacement is 1.43 inches, the roof displacement is 1.93 inches, and the period is 0.97 second (refer to sheet 20).

A new mathematical model is constructed that allows the base of two columns to yield in flexure. A nominal lateral force is applied. The relative distribution of beam moments will vary from the distribution of beam moments shown on sheet 10 for seismic forces. New values for periods, mode shapes, and participation factors are calculated. The forces are proportionally adjusted until a number of additional structural elements begin to yield (±5% of calculated yield capacity). At an additional equivalent base shear coefficient of 0.06, yielding occurs at the base of the third (left) column, the tops of the other two first-story columns, the top and bottom of the second-story center column, and the north end of the first- and second-story beams (Model 3 on sheet 19). The period of this revised model is 1.14 seconds and the roof displacement is 1.10 inches for the base shear of 0.06. When the results of this model are superimposed on the initial model, the following results are obtained: base shear is 0.19 (0.13 + 0.06), spectral acceleration is 0.224g, spectral displacement is 2.27 inches, the roof displacement is 3.02 inches, and the effective period is 1.02 seconds. These results are summarized on sheet 20.

The mathematical model is revised again to allow the newly formed hinges to yield. These hinges were given sectional properties roughly equal to 5% of their fully elastic value. An additional set of periods, mode shapes, and participation factors are calculated. New increments of force are applied until additional hinges form and a mechanism forms at the first floor (see model 4 on sheet 19). The period for this last increment of displacement is 2.29 seconds, the base shear coefficient is 0.04, and the roof displacement is 2.69 sec-inches. When these results are superimposed on the previous results, the following values were obtained: base shear is 0.23, spectral acceleration is 0.257g, spectral displacement is 4.45 inches, roof displacement is 5.71 inches, and the effective period of vibration is 1.33 seconds (refer to sheet 20).

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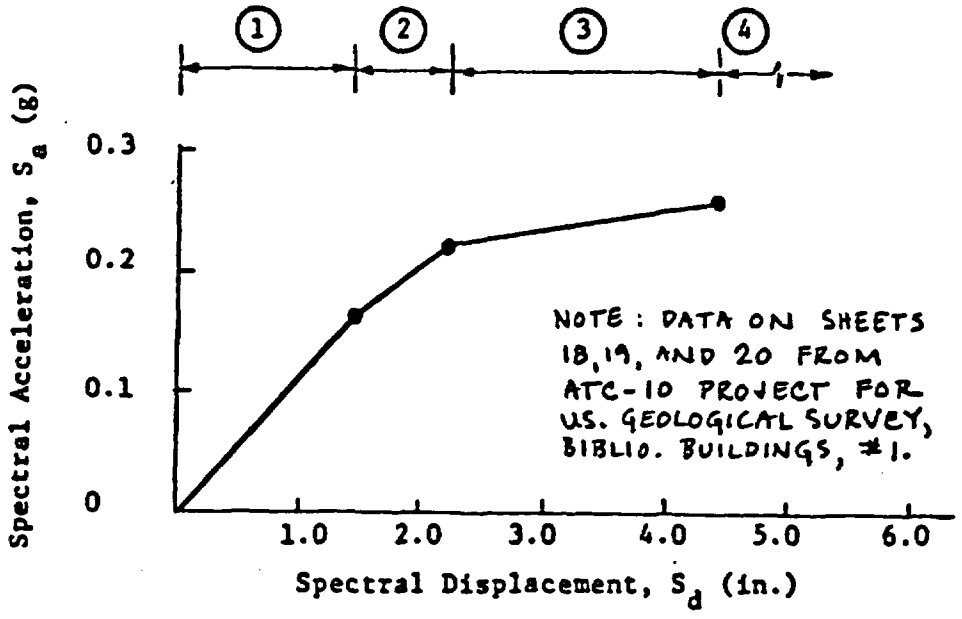
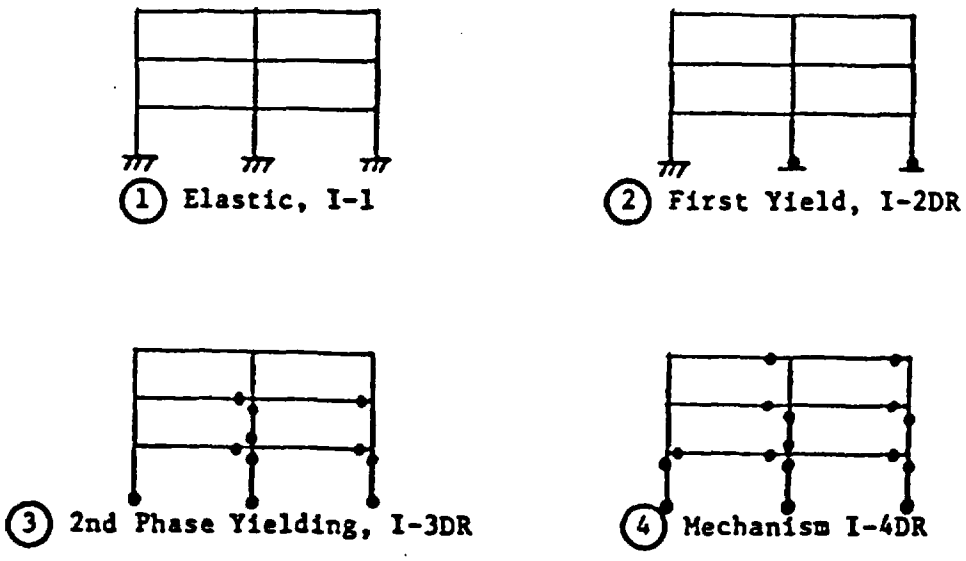
Example E-3	18 of 34	Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.



CAPACITY CURVE



MODELS

METHOD 2 MODELS AND CAPACITIES

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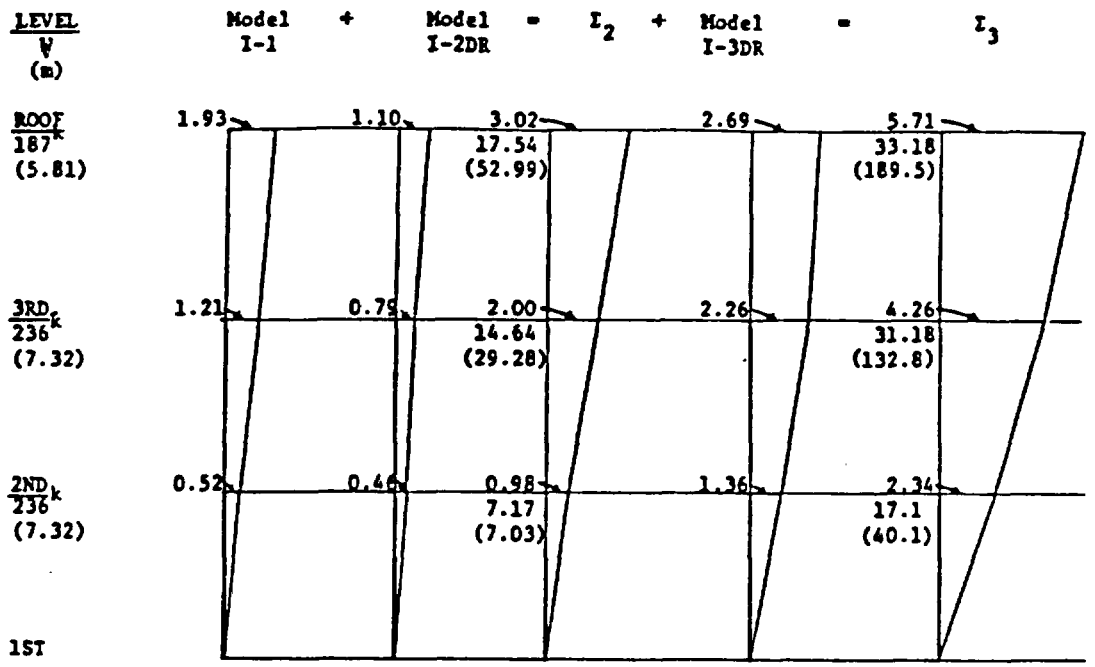
Example E-3 19 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

DEFLECTED SHAPES (INCHES)



W=659k
(Im=20.45)k-Sec²/ft.

$Im\Delta_2$= 39.35
($Im\Delta^2$= 89.3)

$Im\Delta$= 81.46
($Im\Delta^2$=362.4)

Δ_I(in)	1.93	1.10	3.02	2.69	5.71
T(sec)	0.97	1.14		2.29	
C_B	0.13	0.06	0.19	0.04	0.23
C_B/S_a	0.805	0.897	0.848	0.934	0.895
S_a	0.161g		0.224g		0.257g
Δ_I/S_d	1.345	1.298	1.331	1.219	1.283
S_d(in)	1.43		2.27		4.45
$T=2\pi\sqrt{\frac{S_d}{S_a g}}$	0.95		1.02 sec.		1.33 sec.

CAPACITY SUMMARY

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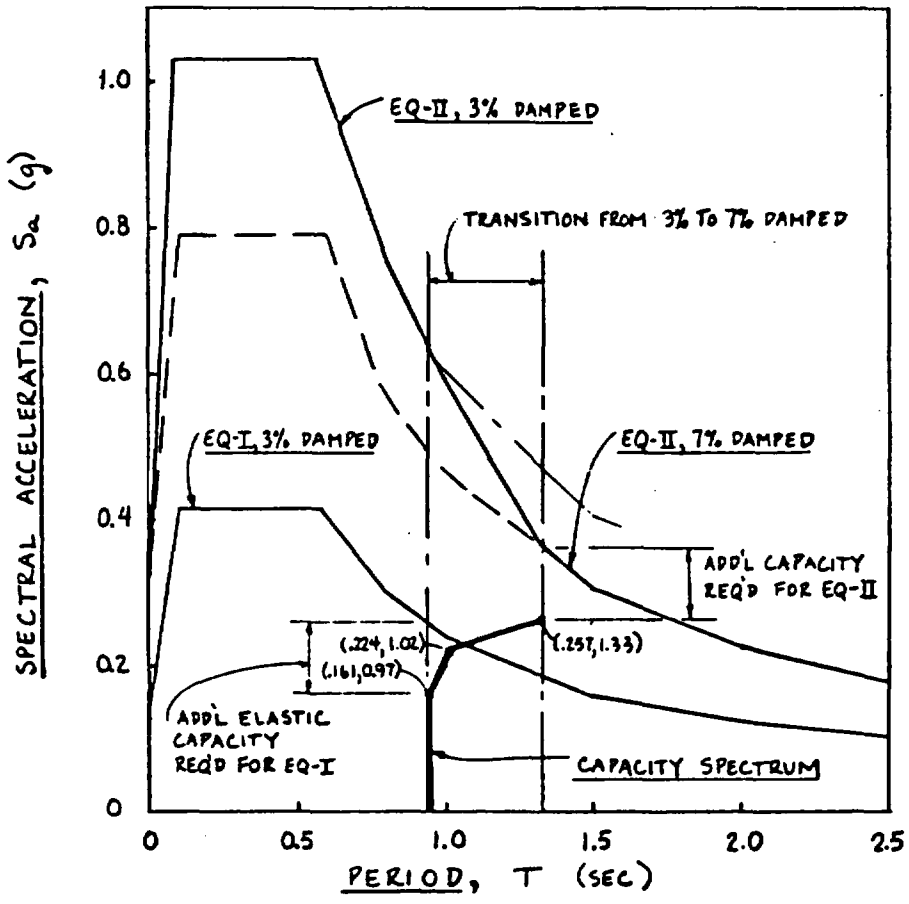
Example E-3 20 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.



REFER TO SHEET 20 FOR CAPACITY SPECTRUM VALUES
SHEET 4 FOR RESPONSE SPECTRA

CAPACITY SPECTRUM METHOD

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Example E-3 21 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

METHOD 2 (CONTINUED)

SUMMARY

1. THE STRUCTURE YIELDS AT EQ-I.

2. THE FIRST YIELD OCCURS AT 0.66 EQ-I; THEREFORE IT DOES NOT HAVE THE CAPACITY TO SATISFY THE NEARLY ELASTIC CRITERIA.

3. THE CAPACITY SPECTRUM DOES NOT CROSS THE EQ-II DEMAND RESPONSE SPECTRUM (SHEET 21); THEREFORE THE STRUCTURE DOES NOT SATISFY THE EQ-II CRITERIA. REFER TO PARAGRAPH 5-5b(2)(g) AND TO FIGURE 5-6.

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Example E-3	22 of 34	Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

FRAME 4 (DMRSF) - MODIFICATIONS

THE TRANSVERSE FRAMES MUST BE MODIFIED IN ORDER TO INCREASE THEIR CAPACITY TO RESIST SEISMIC LOADING. THREE POSSIBLE MODIFICATION SCHEMES ARE DISCUSSED BELOW.

- THE BUILDING ACTUALLY CONTAINS 7 TRANSVERSE FRAMES, ONLY 3 OF WHICH HAVE BEEN DETAILED AS DUCTILE MOMENT RESISTING FRAMES. BY CHANGING THE CONNECTION DETAILS FOR THE 4 INTERMEDIATE FRAMES, ALL 7 FRAMES MAY BE USED TO RESIST THE LATERAL LOADS.

- THE MEMBER SIZES FOR BEAMS AND COLUMNS IN FRAMES 1, 4 AND 7 CAN BE INCREASED TO IMPROVE THEIR LATERAL RESISTANCE. FOR EXAMPLE, THE ROOF BEAM HAS A DEMAND MOMENT $M_D = 199$ k-ft. A SECTION WITH SUFFICIENT PLASTIC CAPACITY MUST HAVE A PLASTIC SECTION MODULUS $Z_x = 199(12)/36 = 66.3$ in^3. A W14x43 ($Z_x = 69.6$) OR A W18x35 ($Z_x = 66.5$) WOULD BE ADEQUATE. AFTER THE MEMBERS HAVE BEEN RESIZED, THE ANALYSES FOR EQ-I AND EQ-II SHOULD BE REPEATED. INCREASING THE STIFFNESS OF THE FRAME MAY RESULT IN A HIGHER 1ST MODE SPECTRAL ACCELERATION AND A HIGHER DESIGN BASE SHEAR.

- THE TRANSVERSE FRAMES CAN BE STRENGTHENED WITH THE ADDITION OF A BRACED FRAME SYSTEM. THIS WILL STIFFEN THE STRUCTURE AND INCREASE THE SEISMIC FORCES SO THAT THE EQ-I AND EQ-II ANALYSES MUST BE REPEATED. SEE THE FOLLOWING SECTION FOR AN EXAMPLE OF A BRACED FRAME ANALYSIS.

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Example E-3 23 of 34 Steel Frames

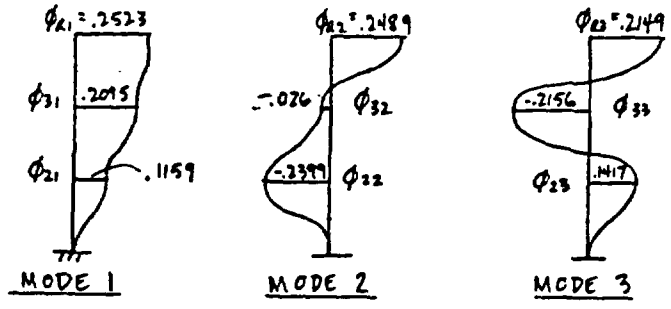
Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

LONGITUDINAL (E-W) DIRECTION: FRAME A - BRACED FRAME

MODE SHAPES ($\phi_{\ell m}$) AND PERIODS (T_m) FROM COMPUTER ANALYSIS OF FRAME A, MASS CALCULATED FROM W/g.

LEVEL	MASS ($\frac{k \cdot sec^2}{ft.}$)	MODE 1			MODE 2			MODE 3		
		$\phi_{\ell 1}$	$m_\ell \phi_{\ell 1}$	$m_\ell \phi_{\ell 1}^2$	$\phi_{\ell 2}$	$m_\ell \phi_{\ell 2}$	$m_\ell \phi_{\ell 2}^2$	$\phi_{\ell 3}$	$m_\ell \phi_{\ell 3}$	$m_\ell \phi_{\ell 3}^2$
R	5.82	.2523	1.468	.370	.2489	1.449	.361	.2149	1.251	.269
3	10.98	.2095	2.300	.482	-.0262	-.288	.008	-.2156	-2.367	.510
2	10.98	.1159	1.273	.147	-.2399	-2.634	.632	.1417	1.556	.220
Σ	27.78		5.041	.999		-1.473	1.001		.440	.999
$PF_{\ell m}$* (Eq. 4-1)		$\frac{\Sigma m \phi}{\Sigma m \phi^2 \phi_{\ell 1}}$ =		1.273		-.366			.095	
PF_{3m}				1.057		.039			-.095	
PF_{2m}				.585		.353			.062	
α_m (Eq. 4-2)		$\frac{(\Sigma m \phi)^2}{\Sigma m (\Sigma m \phi^2)}$ =		.916		.078			.007	
T_m, sec			.299			.112			.079	

* AS A CHECK, NOTE THAT $\sum_{m=1}^{3} PF_{\ell m} = 1.0$ AND $\sum_{m=1}^{3} \alpha_m = 1.0$.



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Example E-3 24 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

LONGITUDINAL (E-W) DIRECTION: SPECTRAL ACCELERATIONS, S_{am}
MODAL BASE SHEARS, V_m

EQ		MODE 1	MODE 2	MODE 3
EQ-I (β=3%)	T_m, sec (SHEET 24)	.299	.112	.079
	S_{am}, g (SHEET 4)	.41	.41	.41
	$C_{bm}=\alpha_m S_{am}$	.376	.032	.003
	$V_m=C_{bm}W$ (Eq. 4-4)	335.6 k	28.6 k	2.55 k
EQ-II (β=7%)	$T_m\sqrt{1.0}$*, sec	.299	.112	.079
	S_{am}, g	.79	.79	.79
	$C_{bm}=\alpha_m S_{am}$	.724	.062	.006
	$V_m'=C_{bm}W$	646.6 k	55.1 k	4.91 k

* NOTE: AS SHOWN IN TABLE 4-2, THE INELASTIC DEMAND RATIO FOR K-BRACES IN A CRITICAL, ESSENTIAL FACILITY IS 1.0. THUS, THE LONGITUDINAL PERIOD IS THE SAME FOR BOTH EQ-I AND EQ-II AND THE SAME COMPUTER MODEL WAS USED FOR BOTH ANALYSES.

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Example E-3 25 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

MODAL ANALYSIS: LONGITUDINAL (E-W) DIRECTION - FRAME A

		LEVEL	PF_{xm}	$\frac{m_x \phi_{xm}}{\Sigma m_x \phi_{xm}}$	F_{xm} (k)	V_{xm} (k)	ΔOTM_{xm} (ft-k)	OTM_{xm} (ft-k)	$a_{xm} = \frac{F_{xm}}{W_x}$	δ_{xm} (in.)	Δ_{xm} (in)
MODE 1	EQ-I	R	1.273	.291	97.7	97.7	1075	0	.522	.456	.078
		3	1.057	.456	153.2	250.9	2760	1075	.433	.378	.169
		2	.585	.253	84.7	335.6	3692	3835	.240	.209	.209
				1.000				7526			
	EQ-II	R			188.3	188.3	2071	0	1.005	.878	.150
		3			295.0	483.3	5316	2071	.834	.728	.324
		2			163.3	646.6	7113	7388	.462	.404	.404
								14500			
MODE 2	EQ-I	R	-.366	-.984	-28.2	-28.2	-310	0	-.150	-.018	.020
		3	.039	.196	5.6	-22.6	-249	-310	.016	.002	.016
		2	.353	1.788	51.2	28.6	315	-559	.145	.018	.018
				1.000				-244			
	EQ-II	R			-54.3	-54.3	-597	0	-.290	-.036	.040
		3			10.8	-43.5	-479	-597	.030	.004	.030
		2			98.6	55.1	606	-1076	.279	.034	.034
								-470			
MODE 3	EQ-I	R	.095	2.843	7.26	7.26	80	0	.039	.0024	.005
		3	-.095	-5.380	-13.74	-6.48	-71	80	-.039	-.0024	.004
		2	.062	3.537	9.03	2.55	28	9	.026	.0016	.002
								37			
	EQ-II	R			14.0	14.0	154	0	.075	.0046	.009
		3			-26.5	-12.5	-138	154	-.075	-.0046	.008
		2			17.41	4.91	54	17	.049	.0030	.003
								71			
SRSS	EQ-I	R			102.0	102.0	1078	0	.544	.456	.081
		3			153.9	251.9	2772	1078	.435	.378	.170
		2			99.4	336.8	3706	3876	.281	.210	.210
								7530			
	EQ-II	R			196.5	196.5	2161	0	1.048	.878	.156
		3			296.4	485.5	5339	2161	.838	.729	.325
		2			191.6	649.0	7139	7466	.542	.405	.405
								14508			

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Example E-3 26 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

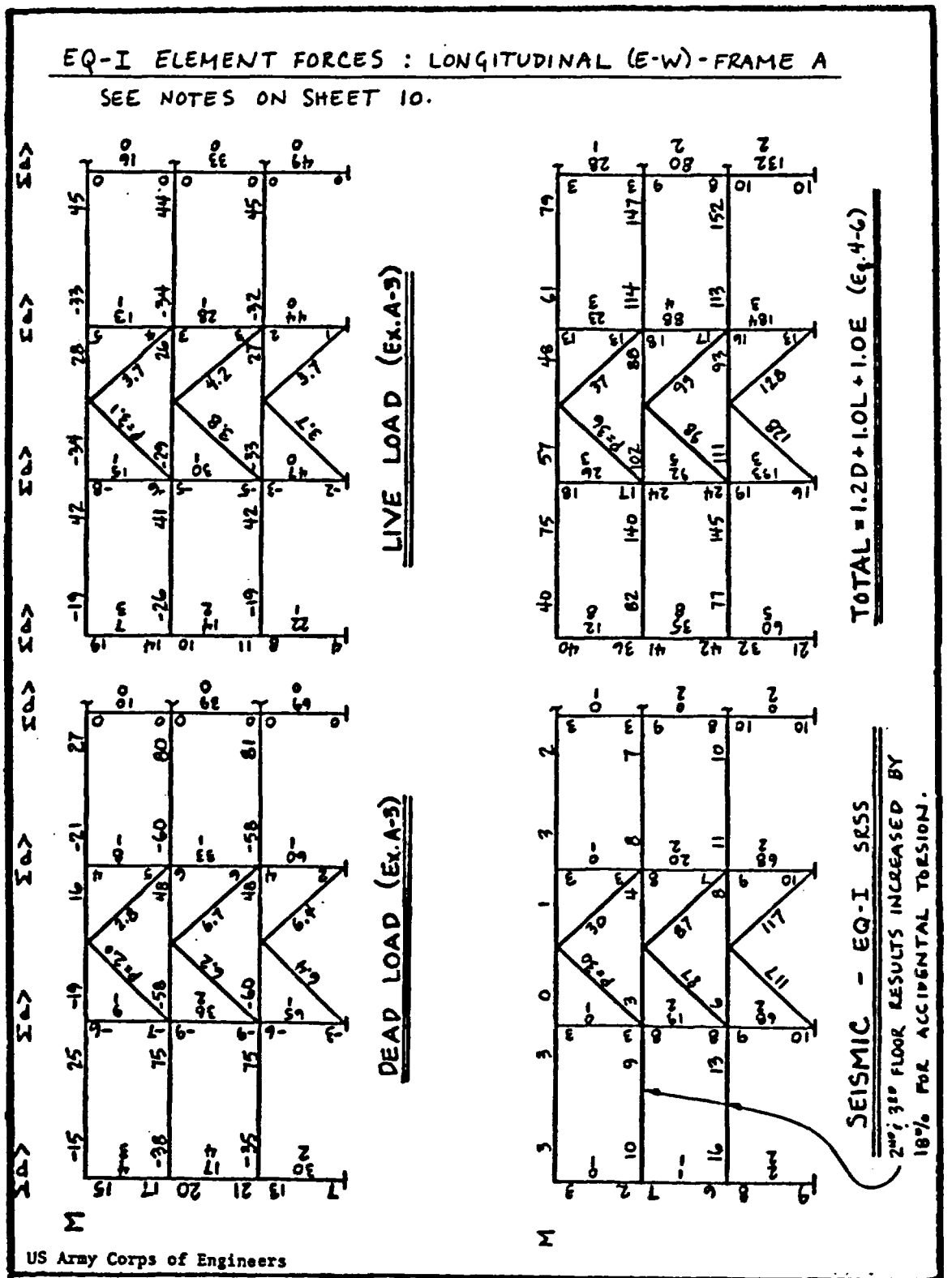

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

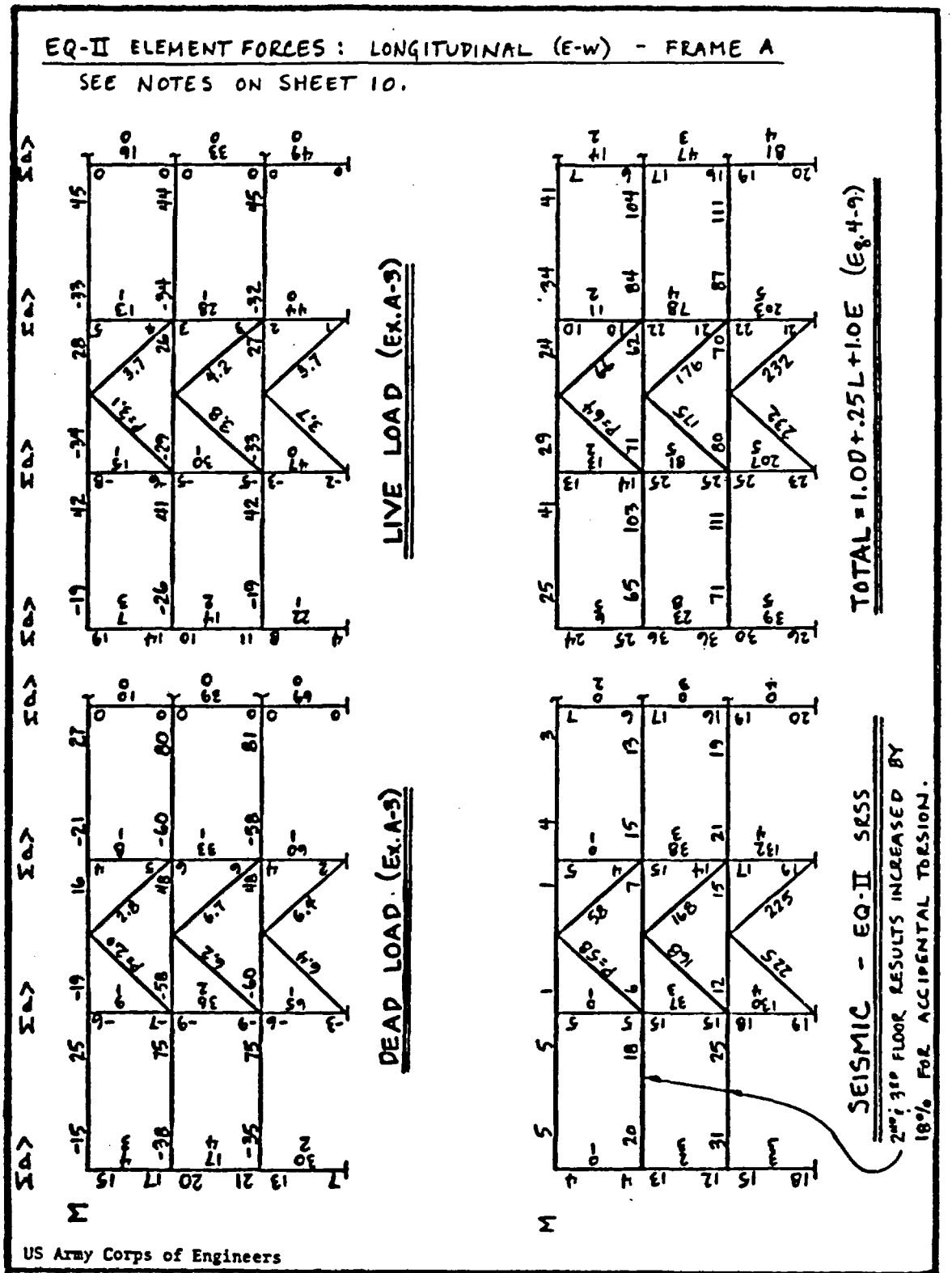

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

ELEMENT STRESSES - FRAME A (BRACED FRAME)

• K-BRACES 1ST STORY (F_y = 46 ksi)

STEEL TUBE	A (in²)	r (in)	kL/r	F_a	P_{cr} = 1.7AF_a (k)	EQ-I P_D (k)	EQ-I P_D/P_{cr}	EQ-II P_D (k)	EQ-II P_D/P_{cr}	IDR*
5×5×¼	4.59	1.92	121.4	10.13	79.1	128	1.62 ×	232	2.93 ×	1.0

• BEAM ELEMENTS IN UNBRACED BAYS

LEVEL	SIZE	Z_x (in³)	EQ-I M_D (k-ft)	EQ-I M_c (k-ft)	EQ-I M_D/M_c	EQ-II M_D (k-ft)	EQ-II M_c (k-ft)	EQ-II M_D/M_c	IDR*
ROOF	W14×30	47.3	79	142	.56	42	142	.30	1.5
2	W18×40	78.4	152	235	.65	111	235	.47	1.5

*INELASTIC DEMAND RATIO FROM TABLE 4-2.

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Example E-3 29 of 34 Steel Frames

Figure E–3. Building with steel moment-resisting frames and steel braced frames—continued.

ELEMENT STRESSES - FRAME A (CONTINUED)

• BEAM ELEMENTS IN BRACED BAYS (CHECK INTERACTION EQUATIONS, SAME AS FOR COLUMNS) L=16'

EQ-I EC ≥ 1.2D + 1.0L + 1.0E

SIZE	A (in²)	S_x (in³)	EQ-I P_D (k)	M_D (k-ft)	f_a (ksi)	f_{bx} (ksi)	F_a (ksi)	F_{bx} (ksi)	F'_{ex} (ksi)	* EQ1.6-1a	* EQ1.6-1b
W14×30	8.85	42.0	29.7	57	3.36	16.3	19.7	24.0	133	.34	.49
W18×40	11.8	68.4	105.5	111	8.94	19.5	20.2	19.1	211	.64	.84

* MODIFIED INTERACTION EQUATIONS - SEE FRAME 4, SHEET 14.

EQ-II UC ≥ 1.0D + .25L + 1.0E

SIZE	A (in²)	Z_x (in³)	EQ-II P_D (k)	M_D (k-ft)	M_D (k-in)	P_y (k)	P_{cr} (k)	M_{px} (k-in)	M_{pcx} (k-in)	M_x † / M_{pcx}	IDR (TABLE 4-2)
W14×30	8.85	47.3	52.7	29	348	318.6	296.4	1703	1677	.21	1.5
W18×40	11.8	78.4	191.1	80	960	424.8	405.2	2822	1832	.52	1.5

† SEE FIGURE 4-2, $M_x/M_{pcx} \leq \mu$ ... (μ = IDR)

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Example E-3 30 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

ELEMENT STRESSES - FRAME A (CONTINUED)

• STEEL COLUMNS 1ST STORY @ BASE

EQ-I EC ≥ 1.2D + 1.0L + 1.0E

SIZE	A (in²)	S_y (in³)	EQ-I P_D (k)	M_D (k·ft)	f_a (ksi)	f_{by} (ksi)	F_a (ksi)	F_{by} (ksi)	F'_{ey} (ksi)	* EQ1.6-1a	* EQ1.6-1b
W14×43	12.6	11.3	193	16	15.3	17.0	17.1	27	37.6	.72	.79
W14×48	14.1	12.8	132	10	9.36	9.38	17.2	27	38.5	.42	.46

*MODIFIED UNIAXIAL INTERACTION EQUATIONS - SEE FRAME 4, SHEET 14.

EQ-II UC ≥ 1.0D + .25L + 1.0E

SIZE	A (in²)	Z_y (in³)	EQ-II P_D (k)	M_D (k·ft)	M_D (k·in)	P_y (k)	P_{cr} (k)	M_{py} (k·in)	M_{pcy} (k·in)	M_y † / M_{pcy}	IDR (TABLE 4-2)
W14×43	12.6	17.3	207	23.	276	453.6	366.3	623	587	.47	1.5
W14×48	14.1	19.6	81	20	240	507.6	412.3	706	819	.29	1.5

† SEE FIGURE 4-2, $M_y/M_{pcy} \leq \mu$. (μ = IDR)

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Example E-3 31 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

CHECK INTERSTORY DRIFT

ALLOWABLE DRIFT FOR ESSENTIAL FACILITIES:
.005h FOR EQ-I
.010h FOR EQ-II

LONGITUDINAL (E-W) DIRECTION - FRAME A

LEVEL	STORY HT. h, in	EQ-I		EQ-II	
		*Δ_{SESS}, in	.005h	*Δ_{SESS}, in	.010h
ROOF	132	.081	.66	.156	1.32
3	132	.201	.66	.384	1.32
2	132	.248	.66	.478	1.32

* Δ_{SESS} FROM PREVIOUS CALCULATIONS ON SHEET 26 SCALED BY 1.18 AT THE 2ND & 3RD FLOORS TO ACCOUNT FOR ACCIDENTAL TORSION.

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Example E-3 32 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

FRAME A - COMMENTARY

EQ-I THE ELASTIC ANALYSIS INDICATES THAT THE BRACES IN THE FIRST STORY ARE OVERSTRESSED BY 62%, ALTHOUGH THE BEAMS AND COLUMNS AT THIS LEVEL ARE NOT OVERSTRESSED. THE FRAME HAS EXCEEDED THE 10% OVERSTRESS ALLOWED FOR SOME MEMBERS AT EACH STORY AND THEREFORE DOES NOT MEET THE "NEARLY ELASTIC" CRITERIA FOR A K=1.00 FRAME.

EQ-II NO OVERSTRESS IS ALLOWED FOR K-BRACE ELEMENTS IN AN ESSENTIAL FACILITY SINCE THIS TYPE OF STRUCTURAL SYSTEM HAS LITTLE RESERVE CAPACITY ONCE THE COMPRESSION BRACES HAVE BUCKLED. THE FIRST STORY BRACES ARE OVERSTRESSED BY 193% FOR EQ-II, ALTHOUGH ALL OF THE OTHER ELEMENT STRESSES ARE WITHIN ALLOWABLE LIMITS.

CONCLUSION SINCE THE FRAME DOES NOT MEET THE REQUIREMENTS OF EITHER EQ-I OR EQ-II, THE BRACING SYSTEM MUST BE MODIFIED.

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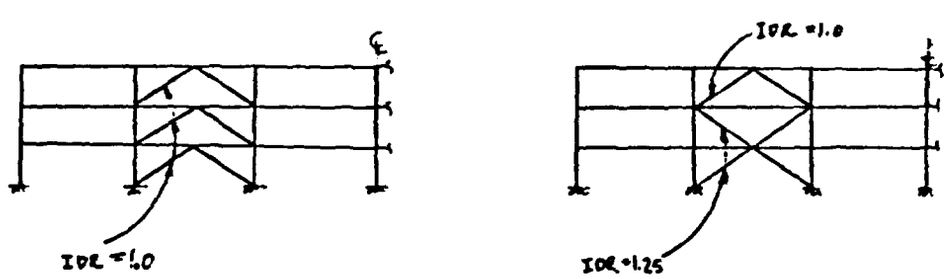
Example E-3 33 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

FRAME A - MODIFICATIONS

THE LONGITUDINAL FRAMES REQUIRE MODIFICATION IN ORDER TO INCREASE THEIR SEISMIC RESISTANCE. THE K-BRACES AND BEAMS IN THE BRACED BAYS OF THE LOWER TWO STORIES ARE OVERSTRESSED FOR EQ-I. IN ADDITION TO INCREASING THE MEMBER SIZES FOR THE ELEMENTS WHICH ARE OVERSTRESSED, IT IS ALSO ADVANTAGEOUS TO CHANGE THE GEOMETRY OF THE BRACES IN THIS FRAME. TABLE 4-2 SHOWS AN INELASTIC DEMAND RATIO OF 1.0 FOR K-BRACES IN AN ESSENTIAL FACILITY, BUT 1.25 FOR DIAGONAL BRACES. THUS, REVERSING THE ORIENTATION OF THE K-BRACE IN THE 2ND STORY WOULD RESULT IN A DIAGONAL BRACING SCHEME FOR THE LOWER FLOORS WHERE THE BRACES ARE OVERSTRESSED. THESE DIAGONAL BRACES MUST BE SIZED TO REMAIN ELASTIC FOR EQ-I BUT ARE ALLOWED 25% OVERSTRESS FOR EQ-II.



FRAME A

MODIFIED FRAME A

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Example E-3 34 of 34 Steel Frames

Figure E-3. Building with steel moment-resisting frames and steel braced frames—continued.

DESIGN EXAMPLE: E-4

SEVEN-STORY DUCTILE CONCRETE FRAME BUILDING:

Purpose. This example is presented in order to illustrate the modal analysis of a multistory building and the procedure for checking the ductility of beams and columns in a reinforced concrete frame.

Description of Structure. Design example E-4 is based upon a building with the same characteristics as the one that was used for design example E-1 and for the examples given in paragraph 2-5c of this manual. The building is a 7-story, reinforced concrete moment-resisting space frame building as shown on sheet 2. The computer program TABS was used to model the structure for the seismic analyses. The section properties for the model were based on gross concrete sections and the properties for the spandrel beams around the perimeter were increased by 50% to approximate the influence of the slab.

Modal Analysis. The transverse modal analysis of the structure is shown in example E-1. The site response spectra for EQ-I and EQ-II were provided by the soils engineer. The spectrum for EQ-I was based on 5% structural damping and a soil profile similar to type S_2. The EQ-I spectrum has a peak ground acceleration of 0.20g and a maximum spectral acceleration of 0.50g. The seismic analyses included three modes of vibration from which the SRSS responses were determined.

Ductility Check. One beam and one column section were selected from the sixth-floor level of frame B in order to illustrate the ductility check procedure. The properties of these sections and appropriate dead load, live load, and seismic analysis results are shown on sheets 5 and 6. The beam ductility check is presented on sheets 6-8 and the column ductility check is on sheets 9-12.

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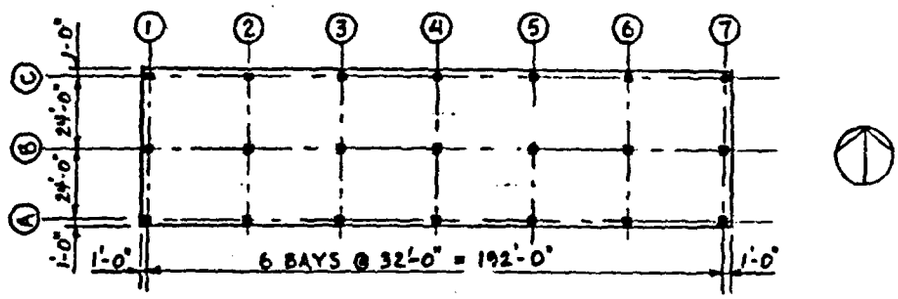
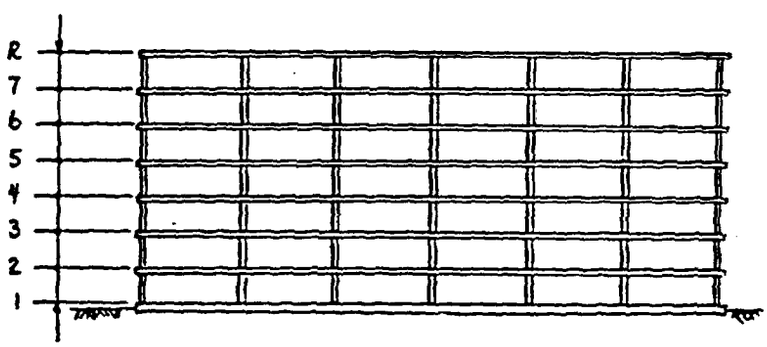
Example E-4 1 of 12 Concrete Frame

Figure E-4. Seven-story ductile concrete frame building.

7-STORY REINFORCED CONCRETE FRAME BUILDING



PLAN



SOUTH ELEVATION

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Example E-4 2 of 12 Concrete Frame

Figure E-4. Seven-story ductile concrete frame building—continued.

MODAL ANALYSIS - INFLUENCE OF HIGHER MODES (SEE PARA. 5-4a(3))

HIGHER MODES OF RESPONSE BECOME MORE IMPORTANT IN THE RESPONSE OF THE UPPER STORIES OF A MULTI-STORY BUILDING. A COMPARISON OF THE MODAL STORY SHEARS AND THE SRSS STORY SHEARS IS SHOWN BELOW.

STORY SHEARS, EQ-I

LEVEL	V_{SRSS}	MODE 1			MODE 2		MODE 3	
		V_1*	V_1/V_{SRSS}	$(V_1/V_{SRSS})^2$	V_2*	$(V_2/V_{SRSS})^2$	V_3*	$(V_3/V_{SRSS})^2$
R	629	508	.81	.65	-330	.275	170	.073
7	1139	1002	.88	.774	-518	.207	160	.020
6	1529	1445	.95	.893	-499	.107	-6	—
5	1846	1816	.98	.968	-283	.024	-169	.008
4	2106	2098	.996	.992	46	—	-175	.007
3	2312	2283	.987	.975	365	.025	-19	—
2	2498	2408	.964	.929	632	.064	200	.006

* MODAL SHEARS FROM DESIGN EXAMPLE E-1, SHEETS 4,5 & 6.

THUS, FOR THIS REGULAR 7-STORY CONCRETE FRAME BUILDING, THE 2ND AND 3RD MODES CONTRIBUTE VERY LITTLE TO THE STORY SHEARS AT FLOORS 2-5. FOR FLOORS 6,7, & R, THESE HIGHER MODES CONTRIBUTE 11%, 23%, AND 35% RESPECTIVELY, WHEN THE MODAL FORCES ARE COMBINED ON AN SRSS BASIS.

THE SRSS RESULTS FROM THE 3-MODE ANALYSIS WILL BE USED TO CHECK THE BEAM AND COLUMN IN THE REMAINDER OF THIS EXAMPLE.

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Example E-4 3 of 12 Concrete Frame

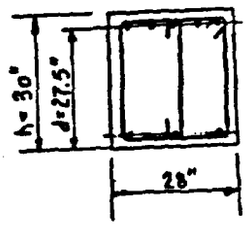
Figure E-4. Seven-story ductile concrete frame building—continued.

ELEMENT PROPERTIES

FOR THIS EXAMPLE, ONE BEAM AND ONE COLUMN WILL BE CHECKED FOR EQ-I AND EQ-II LOADING IN ORDER TO SHOW THE PROCEDURE. THE PROPERTIES OF THE SELECTED ELEMENTS ARE SHOWN BELOW FOR THE INITIAL TRIAL DESIGN.

BEAM

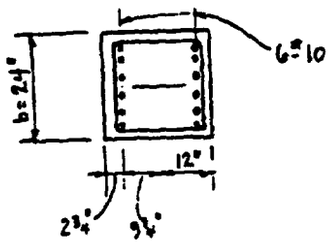
FRAME (B) FLOOR 6 FROM (1) TO (2)
CLEAR SPAN $\ell' = 30.0'$
$f_y = 60$, $f'_c = 4$



	END (1)	MIDSPAN	END (2)
TOP BARS	5-#9		6-#9
BOTTOM BARS	3-#9	4-#9	3-#9

COLUMN

LINE B-2 BETWEEN 5TH & 6TH FLOORS
CLEAR STORY HEIGHT $L_u = 9.5'$
$f_y = 60$, $f'_c = 4$



$A_s = A_s' = 7.62\ in^2$
$d = 21'4"$
$\gamma = 18'2/24 = 0.77$

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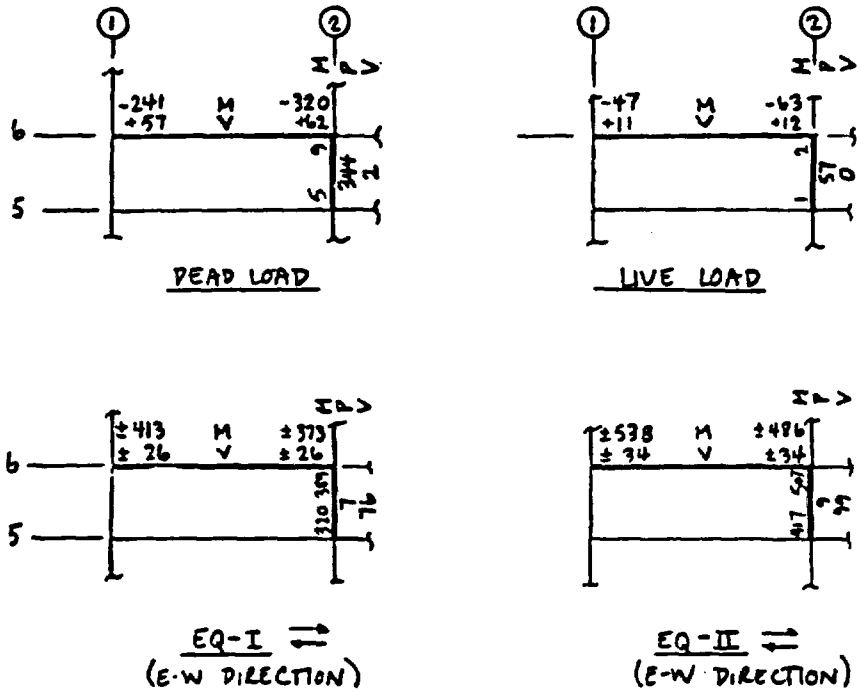
Example E-4	4 of 12	Concrete Frame

Figure E-4. Seven-story ductile concrete frame building—continued.

ELEMENT FORCES (UNITS: k, ft.)

THE RESULTS FROM THE DEAD LOAD, LIVE LOAD AND SEISMIC ANALYSES (NOT INCLUDED) ARE SHOWN BELOW FOR THE SELECTED ELEMENTS IN FRAME B AT THE 6TH FLOOR LEVEL. END MOMENTS AND SHEARS ARE GIVEN FOR THE FACE OF SUPPORT.



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Example E-4 5 of 12 Concrete Frame

Figure E-4. Seven-story ductile concrete frame building—continued.

BEAM FORCES AND LOAD COMBINATIONS (UNITS: k, ft.)

FRAME Ⓑ FLOOR 6 FROM ① TO ②

	END ①		SPAN	END ②	
	M	V	M	M	V
DEAD LOAD (1.0D)	-241	+57	164	-320	+62
LIVE LOAD (1.0L)	-47	+11	31	-63	+12
EQ-I					
SEISMIC (1.0E)	±413	±26	-	±373	±26
1.2D+1.0L+1.0E →	+77	+53	+228	$\boxed{-820}$	+112
1.2D+1.0L+1.0E ←	$\boxed{-749}$	+105	+228	-74	+60
.8D + 1.0E →	+220	+20	+131	-629	+76
.8D + 1.0E ←	-606	+72	+131	+117	+24
EQ-II					
SEISMIC (1.0E)	±538	±34	-	±486	±34
1.0D+.25L+1.0E →	+285	+26	+172	$\boxed{-822}$	+99
1.0D+.25L+1.0E ←	$\boxed{-791}$	+94	+172	+150	+31

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Example E-4 6 of 12 Concrete Frame

Figure E-4. Seven-story ductile concrete frame building—continued.

DUCTILITY CHECK FOR BEAM

THE RATIO OF DEMAND MOMENT TO ULTIMATE MOMENT CAPACITY FOR THE BEAM IS CHECKED AS FOLLOWS:

EQ-I $M_D / M_u < 1.0$

EQ-II $M_D / M_u < 2.5$ (TABLE 4-2)

ULTIMATE MOMENT CAPACITY $M_u \equiv M_c$

$M_u = \phi f_y A_s (d - a/2)/12$ where $\phi = .9$

$f_y = 60$, $f'_c = 4$

$a = A_s f_y / .85 f'_c b$

	END ①	MIDSPAN	END ②
NEGATIVE MOMENT $-M_u$			
TOP BARS	5-#9		6-#9
A_s, in²	5.0		6.0
a, in	3.15		3.78
$-M_u$, k-ft	583		691
POSITIVE MOMENT $+M_u$			
BOTTOM BARS	3-#9	4-#9	3-#9
A_s, in²	3.0	4.0	3.0
a, in	1.89	2.52	1.89
$+M_u$, k-ft	358	472	358

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Example E-4 7 of 12 Concrete Frame

Figure E-4. Seven-story ductile concrete frame building—continued.

DUCTILITY CHECK (CONTINUED) - DEMAND RATIOS

EQ		TOP BARS			BOTTOM BARS			IDR*
		M_D	M_C	$\frac{M_D}{M_C}$	M_D	M_C	$\frac{M_D}{M_C}$	
EQ-I	END ①	749	583	1.28	220	358	.61	1.0
	SPAN	-	-	-	228	472	.48	1.0
	END ②	820	691	1.19	117	358	.33	1.0
EQ-II	END ①	791	583	1.36	285	358	.80	2.0
	SPAN	-	-	-	172	472	.36	2.0
	END ②	822	691	1.19	150	358	.42	2.0

*INELASTIC DEMAND RATIO FOR EQ-II FROM TABLE 4-2.

COMMENT

- FOR EQ-I THE DEMAND MOMENTS HAVE EXCEEDED THE NEGATIVE BENDING CAPACITIES AT BOTH ENDS OF THE BEAM. IF THE TOP STEEL IS INCREASED TO 7-#9 AT BOTH ENDS, THE MOMENT CAPACITY WOULD INCREASE TO 797 k-ft. AND THE DEMAND RATIOS WOULD DECREASE TO .94 AT END ①, AND 1.03 AT END ②. THE CRITERIA FOR "NEARLY ELASTIC" BEHAVIOR (SEE PARA. 4-3e(1)(a)) ALLOW SOME OVERSTRESS, BUT ALL OF THE OTHER BEAMS AT THIS LEVEL MUST BE CHECKED IN ORDER TO DETERMINE WHETHER THE OVERSTRESS AT END ② IS WITHIN ALLOWABLE LIMITS (i.e. 20% OF ALL BEAMS AT THIS FLOOR ARE ALLOWED UP TO 25% OVERSTRESS).

- FOR EQ-II THE DEMAND RATIOS ARE WITHIN THE ALLOWABLE INELASTIC DEMAND CRITERIA IN TABLE 4-2, BUT THE BUILDING AS A WHOLE MUST BE CHECKED FOR MECHANISMS AND UNSYMMETRICAL YIELDING (SEE PARA. 4-4c(5)).

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Example E-4 8 of 12 Concrete Frame

Figure E-4. Seven-story ductile concrete frame building—continued.

COLUMN FORCES AND LOAD COMBINATIONS (UNITS: K, ft.)

COLUMN B-2 BETWEEN 5TH & 6TH FLOORS
E-W EARTHQUAKE

	P	Mx		My*	
		TOP	BOTTOM	TOP	BOTTOM
DEAD LOAD (1.0D)	344	-9	-5	-9	-5
LIVE LOAD (1.0L)	57	-2	-1	-2	-1
EQ-I					
SEISMIC (1.0E)	±7	±389	±320	±78	±64
1.2D+1.0L+1.0E →	463	$\boxed{-402}$	-327	$\boxed{-91}$	-71
1.2D+1.0L+1.0E ←	$\boxed{477}$	376	313	65	57
.8D+1.0E →	268	-396	-324	-85	-68
.8D+1.0E ←	282	382	316	71	60
EQ-II					
SEISMIC (1.0E)	±9	±507	±417	±101	±83
1.0D+.25L+1.0E →	349	$\boxed{-517}$	-422	$\boxed{-111}$	-88
1.0D+.25L+1.0E ←	$\boxed{367}$	498	412	92	78

* IN A REGULAR FRAME BUILDING, AN EARTHQUAKE APPLIED IN THE PLANE OF A GIVEN FRAME WOULD PRODUCE A VERY SMALL OUT-OF-PLANE MOMENT IN AN INTERIOR COLUMN. FOR THE PURPOSE OF ILLUSTRATING A BIAXIAL CHECK, THE SEISMIC MOMENT M_y IS TAKEN AS 20% OF M_x. AN ADDITIONAL BIAXIAL CHECK MUST BE PERFORMED FOR THIS COLUMN FOR LOADS DUE TO A N-S EARTHQUAKE (NOT SHOWN).

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Example E-4 9 of 12 Concrete Frame

Figure E-4. Seven-story ductile concrete frame building—continued.

DUCTILITY CHECK FOR CONCRETE COLUMNS

THE BIAXIAL CAPACITY OF COLUMN B-2 WILL BE CHECKED FOR LOADS RESULTING FROM AN EARTHQUAKE APPLIED IN THE E-W DIRECTION. FIGURE 4-3 PRESENTS THE EQUATIONS REQUIRED FOR A BIAXIAL DUCTILITY CHECK OF A CONCRETE COLUMN. TABLE 4-2 LISTS AN INELASTIC DEMAND RATIO OF 1.25 FOR COLUMNS IN A CONCRETE DMRSF IN AN ESSENTIAL BUILDING.

COLUMN B-2 IS IN COMPRESSION FOR ALL LOAD COMBINATIONS FOR BOTH EQ-I AND EQ-II.

∴ MUST CHECK $\frac{M_x}{M_{ux}}\left(\frac{1-\beta}{\beta}\right) + \frac{M_y}{M_{uy}} \leq \mu_{allow}$

$$\frac{M_x}{M_{ux}} + \frac{M_y}{M_{uy}}\left(\frac{1-\beta}{\beta}\right) \leq \mu_{allow}$$

where μ_{allow} = 1.0 FOR EQ-I
= 1.25 FOR EQ-II (TABLE 4-2)

M_{ux}, M_{uy} = UNIAXIAL ULTIMATE MOMENT CAPACITIES FROM INTERACTION DIAGRAMS

FIND M_{ux}, M_{uy} - UNIAXIAL ULTIMATE MOMENT CAPACITIES

USE INTERACTION DIAGRAMS FROM ACI SP17A-78

FIND $\frac{P_u}{A_g}$: $\frac{477}{24 \times 24} = .83$ FOR EQ-I , $\frac{367}{24 \times 24} = .64$ FOR EQ-II

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Example E-4 10 of 12 Concrete Frame

Figure E-4. Seven-story ductile concrete frame building—continued.

DUCTILITY CHECK - COLUMN B-2 (CONTINUED)

FOR THIS COLUMN $f'_c = 4$ ksi
$f_y = 60$ ksi
$\gamma = 0.77$
$\rho = 15.24/24 \times 24 = .0264$
$M_{ux} = \phi M_{nx} = \left(\frac{\phi M_{nx}}{A_g \times h}\right) \times A_g \times h$, k-ft.

	EQ-I	EQ-II	ACI DIAGRAM
$\phi P_n / A_g$	.83	.64	
$\phi M_{nx}/A_g h$ $M_{ux} = \phi M_{nx}$	.67 772	.64 737	E4-60.75
$\phi M_{ny}/A_g h$ $M_{uy} = \phi M_{ny}$	.48 553	.47 541	L4-60.75

FIND β (SEE PCA BULLETIN 20, "BIAXIAL AND UNIAXIAL CAPACITY OF RECTANGULAR COLUMNS", OR ACI SP17A(78) - COLUMNS II)

$q = \rho f_y / f'_c = .0264(60/4) = .397 \sim .4$

$P_o = (.85 + q) f'_c bt = (.85 + .397)4 \times 24 \times 24 = 2873$ k

$P_u/P_o = 477/2873 = .17$ FOR EQ-I
$367/2873 = .13$ FOR EQ-II

$\beta = .61$ FOR EQ-I
$= .62$ FOR EQ-II
} INTERPOLATE FROM PCA TABLE 5

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Example E-4 11 of 12 Concrete Frame

Figure E-4. Seven-story ductile concrete frame building—continued.

DUCTILITY CHECK - COLUMN B-2 (CONTINUED)

AXIAL COMPRESSION + BIAXIAL BENDING :

$$\frac{M_x}{M_{ux}}\left(\frac{1-\beta}{\beta}\right)+\frac{M_y}{M_{uy}} = \frac{402}{772}\left(\frac{1-.61}{.61}\right)+\frac{91}{553} = .50 < 1.0 \qquad \text{EQ-I}$$

$$\frac{517}{737}\left(\frac{1-.62}{.62}\right)+\frac{111}{541} = .64 < 1.25 \qquad \text{EQ-II}$$

$$\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}\left(\frac{1-\beta}{\beta}\right) = \frac{402}{772}+\frac{91}{553}\left(\frac{1-.61}{.61}\right) = .63 < 1.0 \qquad \text{EQ-I}$$

$$\frac{517}{737}+\frac{111}{541}\left(\frac{1-.62}{.62}\right) = .83 < 1.25 \qquad \text{EQ-II}$$

COMMENT

THIS COLUMN IS ADEQUATE FOR EQ-I AND EQ-II APPLIED IN THE E-W DIRECTION. AN ADDITIONAL CHECK SHOULD BE PERFORMED FOR BIAXIAL BENDING WHEN THE EARTHQUAKE IS APPLIED IN THE N-S DIRECTION.

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Example E-4	12 of 12	Concrete Frame

Figure E-4. Seven-story ductile concrete frame building—continued.

APPENDIX F
DESIGN EXAMPLES—EQUIPMENT IN BUILDINGS

F-1. Purpose and scope.

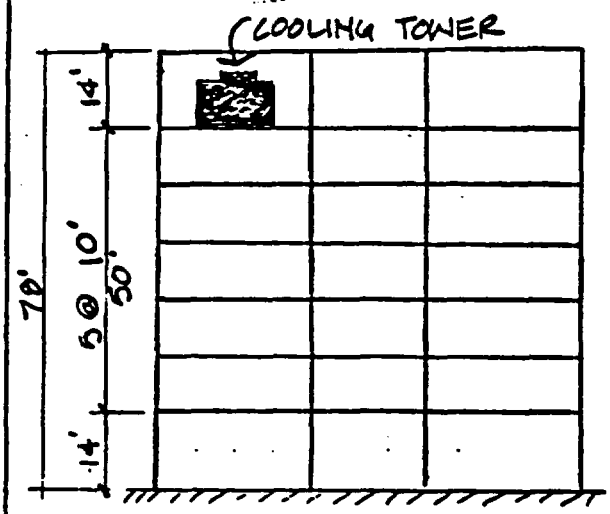
The design examples in this appendix are to illustrate principles, factors, and concepts described in chapter 6 of this manual for the anchorage or bracing of mechanical or electrical equipment in buildings.

F-2. Design examples.

The following design examples are representative of typical mechanical or electrical equipment supported on the roof or on a floor of any building. The various examples illustrate the procedures for the analysis and design of both rigid and flexibly mounted equipment.

Table F-1. Design Examples—Equipment in Buildings.

Fig. No.	*Example No. and Description*
F-1	F-1 Cooling tower in building: presents analysis for a rigidly mounted cooling tower in a multi-story building.
F-2	F-2 Unit heater—flexible brace: analysis of a unit heater not rigidly braced.
F-3	F-3 Unit heater—rigid support: demonstrates the reduction of the lateral seismic load by rigidly bracing the unit heater of design example F-2.
F-4	F-4 Tank on a building: demonstrates the seismic analysis of a storage tank on a building. Emphasis is placed on the period determination.



GIVEN:

WT. OF COOLING TOWER = 20.0 KIPS
WT. TYPICAL FLOOR = 1460 KIPS
CONSIDER TOWER RIGIDLY MOUNTED

EQ-I PER FIG. 2-8, 5% DAMPED BUILDING RESPONSE PER FIG. 2-10

EQ-II IS 2x FIG. 2-8, 10% DAMPED BUILDING PERIODS ARE 40% LONGER THAN EQ-I.
BUILDING ELASTIC CAPACITY RATIO = 1.7

REQUIRED:

FIND SEISMIC DESIGN FORCES FOR EQ-I AND EQ-II TO BE APPLIED AT C.G. OF COOLING TOWER

SOLUTION:

1. EQ-I

FROM FIG. 2-10, MODAL STORY ACCELERATIONS FOR 7TH FLOOR:

	1ST MODE	2ND MODE	3RD MODE	SRSS
a_{xm}	.338	.129	-.007	.362 g

SINCE TOWER IS RIGIDLY CONNECTED,

$$F_p = (a_x)_{MAX} W_p \qquad \text{(EQ. 6-5)}$$

$$F_p = .362\,g \times 20.0^{\text{KIPS}}$$

$$\underline{F_p = 7.24^{\text{KIPS}} \text{ FOR EQ-I}}$$

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Design Example F-1 1 of 2 Cooling Tower in Building

Figure F-1. Cooling tower in building.

2. EQ. II

a) IF ELASTIC CAPACITY OF BUILDING EXCEEDS EQ-II DEMAND, ELASTIC RESPONSE TO EQ-II GOVERNS DESIGN.

$S_a = 2 \times S_{a\,EQ-I}$

$\therefore \; F_p = 2 \times .362 \times 20.0^K$

$= \underline{\underline{14.52^K}}$

b) IF ELASTIC CAPACITY RATIO = 1.7, 2 CONDITIONS RESULT

CONDITION 1: $a_{xm} = .362 \times 1.7 = .615\,g$

CONDITION 2: FIND NEW ACCELERATIONS BASED ON 10% DAMPING AND PERIODS INCREASED 40%

	1ST MODE	2ND MODE	3RD MODE	SRSS
T	1.23 s	.403 sec	.23 s	
$2 \times S_a$	.326	.76	.76	
a_{xm}*	.399	.196	.011	$\underline{\underline{.445}}$

*[FOR a_{xM} ASSUME PROPORTIONAL TO EQ-I ACCELERATIONS

$$a_{xM\,II} = a_{xM\,I}\left(\frac{S_{a\,EQ-II}}{S_{a\,EQ-I}}\right)$$

FOR MODE 1,

$$a_{xM} = .338\left(\frac{.326}{.276}\right) = 0.399$$]

$0.615 > 445 \quad \therefore$ CONDITION 1 GOVERNS

$F_p = 0.615 \times 20^K = \underline{\underline{12.3^K}}$ FOR EQ-II

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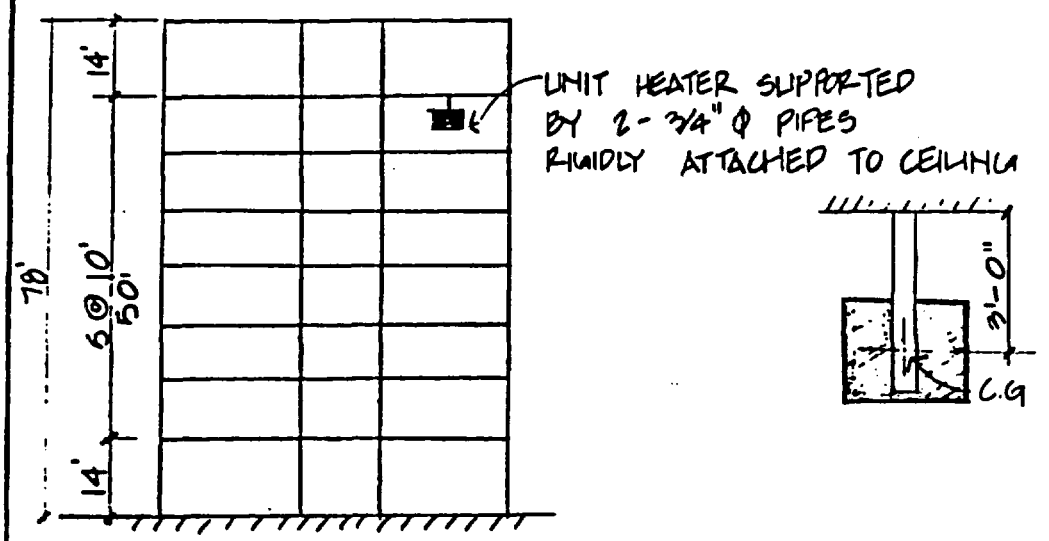
Design Example F-1 | 2 of 2 | Cooling Tower in Building

Figure F-1. Cooling tower in building—continued.



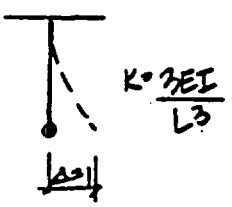
GIVEN: NEGLECT EFFECTS OF ROTATION OF UNIT HEATER.
W_p = WT. OF UNIT HEATER = 350 LB.
EQ-I PER FIG 2-8, 5% DAMPED
BUILDING RESPONSE PER FIG. 2-10
EQ-II IS 2x FIG. 2-8, 10% DAMPED, BUILDING PERIODS ARE 40% LONGER THAN EQ-I.

REQUIRED: FIND SEISMIC DESIGN FORCES FOR EQ-I AND EQ-II TO BE APPLIED AT C.G. OF HEATER.

SOLUTION:

FIND T_a FOR HEATER

$$K = 2\left\{\frac{3(30{,}000)(0.037)}{36^3}\right\} = .142 \text{ K/IN}$$



$$T_a = 0.32\sqrt{\frac{W_p}{K}} = 0.32\sqrt{\frac{.35}{.142}} = \underline{0.50 \text{ SEC}}$$

$T_a > .05$ ∴ QUALIFY AS FLEXIBLE (PARA. 6.3e(1))

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Design Example F-2	1 of 5	Unit Heater - Flexible Brace

Figure F-2. Unit heater—flexible brace.

1. EQ-I
USING FIGS 2-10 AND 6-1,

MODE 1 $T_M = .88$ SEC, $a_{xM} = .338$

T_a/T	0	.5	.8	1.2	2	3
M.F.	1	1	7.5	7.5	1	1
T_a*	0	.44	.704	1.06	1.76	2.64
S_{f_a}**	.338	.338	2.535	2.535	.338	.338

MODE 2 $T_M = .288$, $a_{xM} = .129$

T_a/T	0	.5	.8	1.2	2	3
M.F.	1	1	7.5	7.5	1	1
T_a*	0	.144	.230	.346	.576	.864
S_{f_a}**	.129	.129	.968	.968	.129	.129

* $T_a = T_n (T_a/T)$ (EQ 6-3)
** $S_{f_a} = a_{xm} (M.F.)$ (EQ 6-4)

LOWER LIMIT = $a_{x\,MAX}$ (PARA. 6.3 c (2)(b))

$a_{x\,MAX} = \sqrt{.338^2 + .129^2} = .362$

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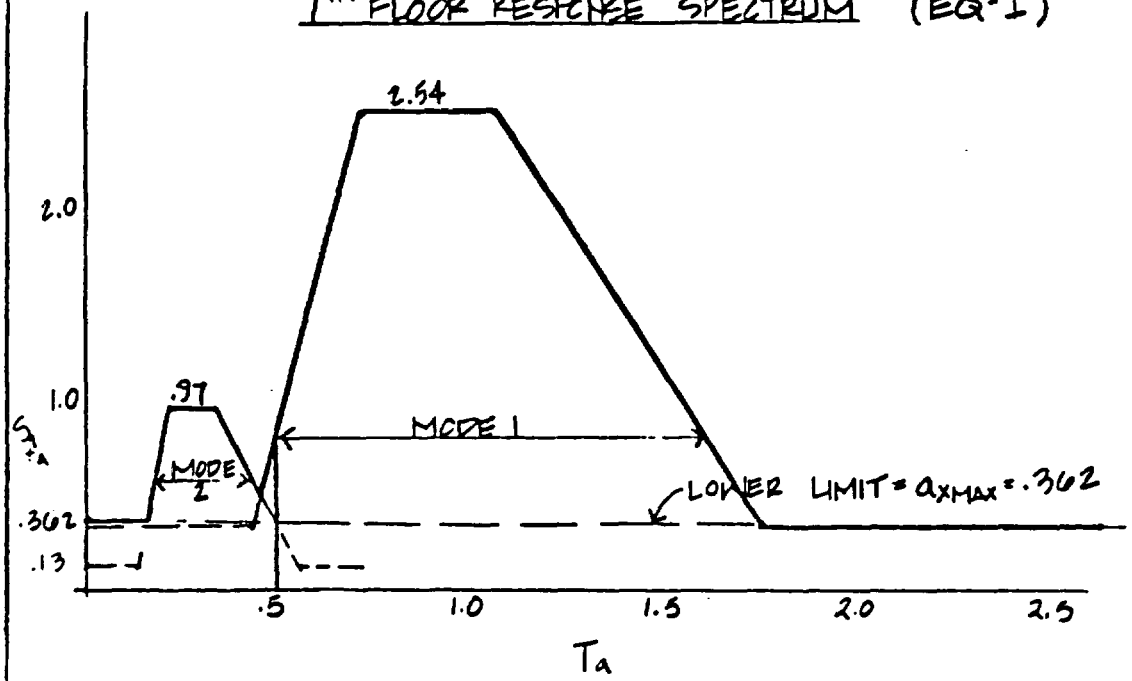
Design Example F-2 2 of 5 Unit Heater - Flexible Brace

Figure F-2. Unit heater—flexible brace—continued.

7TH FLOOR RESPONSE SPECTRUM (EQ-I)



FROM GRAPH, FIND S_{fa} FOR $T_a = .5$

INTERPOLATE:

$$(2.54 - .34)\frac{.5 - .44}{.704 - .44} + .34 = .84$$

FOR FLEXIBLY MOUNTED EQUIPMENT,

$$F_p = S_{fax} W_p \qquad \text{(EQ 6-6)}$$

$$= .84 \times 350^{LB}$$

$F_p = 294$ LBS FOR EQ I

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Design Example F-2	3 of 5	Unit Heater - Flexible Brace

Figure F-2. Unit heater—flexible brace—continued.

2. EQ-II

CONSIDER 2 CONDITIONS, MAXIMUM S_{fa} GOVERNS

CONDITION 1

GIVEN:

ELASTIC CAPACITY RATIO = 1.7

$S_{fa} = 1.7 \times .84 = \underline{1.43}$

CONDITION 2

DRAW POST-YIELD RESPONSE SPECTRUM BASED ON 10% DAMPING, 40% PERIOD INCREASE AND 2 x FIG 2-8 VALUES (SEE F-1)

USE FIG. 6-3.

MODE 1 $T_M = 1.23$, $a_{xm} = .399$

T_a/T	0	.5	.7	1.5	2
M.F.	1.0	1.0	5.0	5.0	1.0
T_a	0	.615	.86	1.85	2.46
S_{fa}	.399	.399	2.0	2.0	.399

MODE 2 $T_M = .403$, $a_{xm} = .196$

T_a/T	0	.5	.7	1.5	2
M.F.	1.0	1.0	5.0	5.0	1.0
T_a	0	.202	.282	.605	.806
S_{fa}	.196	.196	.98	.98	.196

LOWER LIMIT: $a_{xMAX} = \sqrt{.399^2 + .196^2} = .444$

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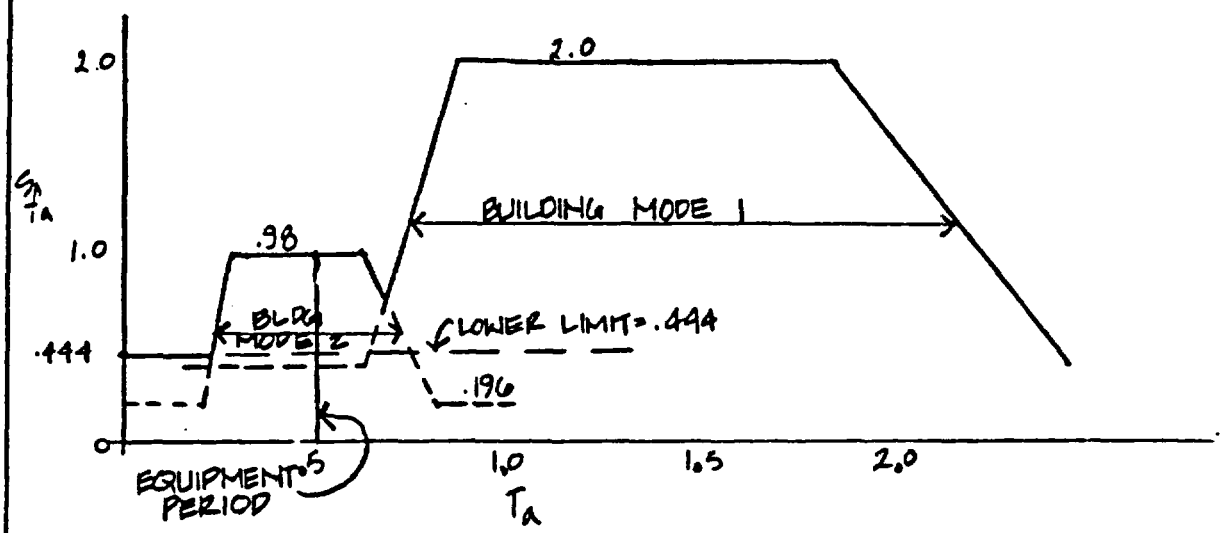
Design Example F-2 4 of 5 Unit Heater - Flexible Brace

Figure F-2. Unit heater—flexible brace—continued.

7TH FLOOR RESPONSE SPECTRUM (EQ-II)



FROM GRAPH, FIND S_{f_a} FOR $T_a = .5$

$\underline{S_{f_a}} = .98$

SINCE 1.43 > .98, <u>CONDITION 1 GOVERNS</u>

$F_p = S_{f_{max}} W_p$ (EQ 6-6)

$= 1.43 \times 350^{LB}$

$= 501$ <u>LBS FOR EQ II</u>

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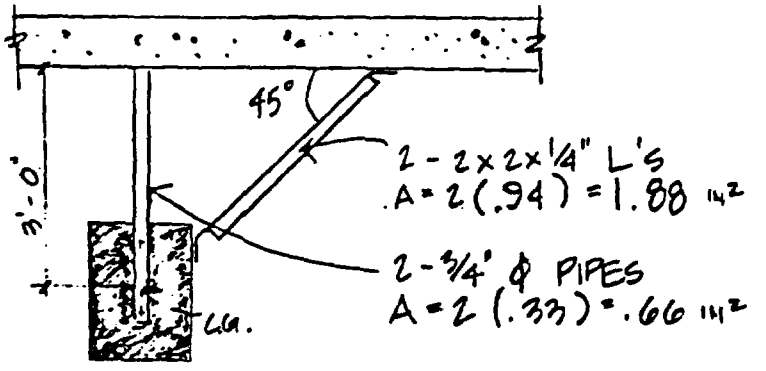
Design Example F-2	5 of 5	Unit Heater - Flexible Brace

Figure F-2. Unit heater—flexible brace—continued.



DETAIL OF UNIT HEATER

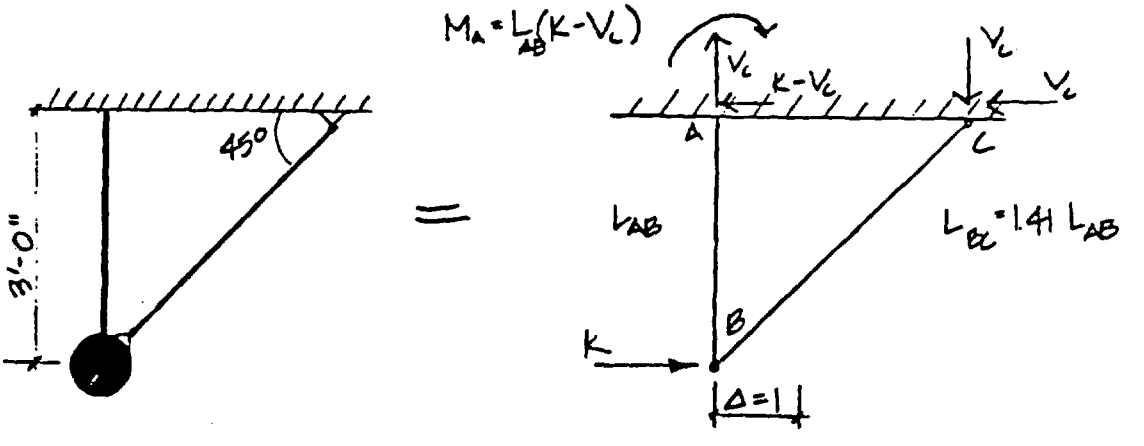
GIVEN: USE DATA GIVEN IN DESIGN EX. F-2
DIAGONAL BRACE ADDED TO STIFFEN SUPPORT

REQUIRED: FIND DESIGN SEISMIC FORCE

SOLUTION:

FIND T_a FOR RIGIDITY CHECK:

APPROXIMATE ANGLE CONNECTIONS BY PINS. ASSUME ALL LATERAL FORCE IS RESISTED BY BRACING ANGLES. USE ENERGY METHOD TO CALC. K_2



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Design Example F-3 1 of 2 Unit Heater - Rigid Support

Figure F-3. Unit heater—rigid support.

ASSUME $K - V_c = 0$ ∴ THIS ASSUMES ALL OF THE HORIZONTAL FORCE K IS RESISTED BY THE DIAGONAL.

$$\Sigma W_{EXTERNAL} = \Sigma W_{INTERNAL}$$

$$K\left(\frac{4}{2}\right) = \frac{K^2 L_{AB}}{2 A_{AB} E} + \frac{(1.41 K)^2 L_{BC}}{2 A_{BC} E}$$

$$1 = K\left(\frac{L_{AB}}{A_{AB} E} + \frac{1.41^3 L_{AB}}{A_{BC} E}\right)$$

$$K = \frac{30 \times 10^6}{\left(\frac{36}{0.66} + \frac{1.41^3 (36)}{1.88}\right)} = 2.78 \times 10^5 \text{ LB/IN}$$

$$T_a = 0.32 \sqrt{\frac{350}{2.78 \times 10^5}} = 0.011 \text{ SEC}$$

$T_a < 0.05$ SEC, THEREFORE SUPPORT IS RIGID (PARA. 6.3e(1))

FIND SEISMIC FORCES - SIMILAR TO F-1

1. EQ-I

$(a_x)_{MAX} = .362$ IN/SEC²

$F_p = a_{xMAX} W_p$

$= .362 \times 350$ LB

$= 126.7$ LB FOR EQ-I

2. EQ-I ELASTIC CAPACITY RATIO = 1.7

CONDITION 1 GOVERNS (SEE F-1)

$a_{xM} = 1.7 \times .362 = .615$ (CONDITION 2, $a_{xMAX} = .444$)

$F_p = a_{xMAX} W_p$

$= .615 \times 350 = 215$ LB FOR EQ-II

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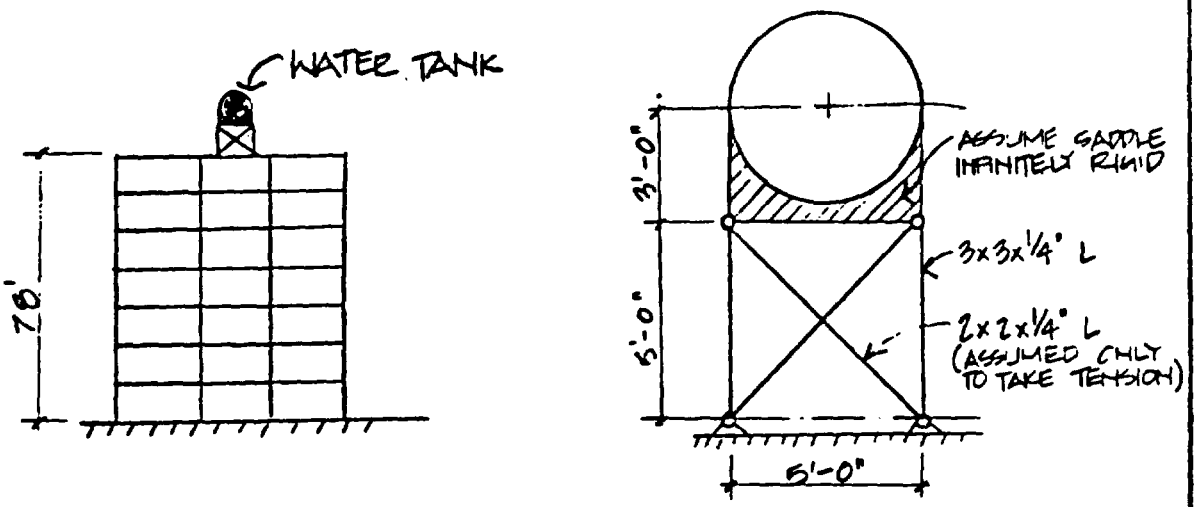
Design Example F-3	2 of 2	Unit Heater - Rigid Support

Figure F-3. Unit heater—rigid support—continued.



DETAIL OF TANK SUPPORT

GIVEN:

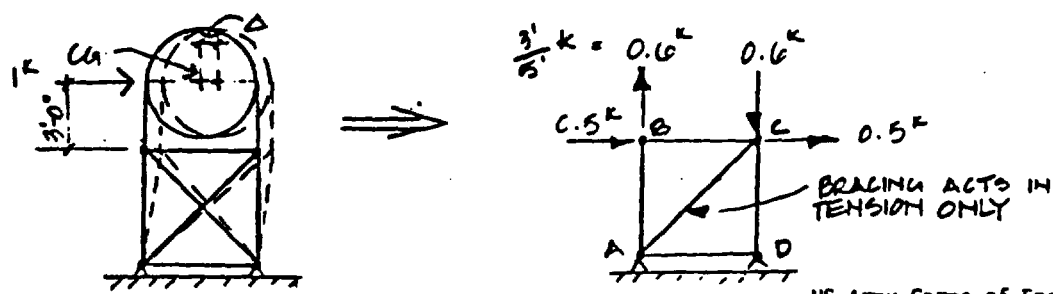
WT OF TANK + WATER = 10.0 K/TRUSS
BUILDING STRUCTURE SAME AS F-1 AND F-2
(SEE FIG. 2-10 FOR BUILDING RESPONSE)
EQ.-I PER FIG. 2-8, 5% DAMPED
EQ-II IS 2 x FIG. 2-8, 10% DAMPED, BUILDING PERIODS 40% LONGER THAN EQ-I.

REQUIRED:

FIND SEISMIC DESIGN FORCES DUE TO EQ-I AND EQ-II.

SOLUTION:

HYDRO-DYNAMIC EFFECTS ARE NEGLECTED EVEN WHEN TANK IS PARTIALLY FULL. CALCULATION OF STIFFNESS OF TANK STRUCTURE: USE ENERGY METHOD TO FIND K.



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Design Example F-4	1 of 6	Tank on a Building

Figure F-4. Tank on a building.

COMPUTATION OF Δ : $1^k \cdot \Delta/2 = \frac{\Sigma F^2 L}{2AE}$

MEMBER	LENGTH	AREA	F	F^2L/A
AB	5.00'	1.44 IN²	+.6^k	1.25
CD	5.00'	1.44	-1.6^k	8.89
CA	7.07'	0.94	+1.414	15.03
				25.17

$$1^k \times \left(\frac{\Delta}{2}\right) = \frac{25.17 \frac{K^2\text{-}FT}{IN^2} \times 12\ IN/FT}{2(30 \times 10^3\ K/IN^2)} = 0.5025 \times 10^{-2}\ IN\text{-}K$$

$$\Delta = 1.005 \times 10^{-2}\ IN/K$$

$$K = 1/\Delta = 99.5\ K/IN$$

$$T_a = .32\sqrt{\frac{W}{K}} = .32\sqrt{\frac{10}{99.5}} = .102\ SEC$$

$T_a > .05$ SEC THEREFORE SUPPORT IS NOT RIGID (PARA. 6.3e(1))

DESIGN AS FLEXIBLY MOUNTED

1. EQ-I

USING FIGS. 2-10 AND 6-1,

MODE 1 $T_M = .88$ SEC, $a_{xM} = .360$

T_a/T	0	.5	.8	1.2	2	3
M.F.	1	1	7.5	7.5	1	1
T_a	0	.44	.704	1.06	1.76	2.64
S_{fa}	.360	.360	2.7	2.7	.36	.36

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Design Example F-4 2 of 6 Tank on a Building

Figure F-4. Tank on a building—continued.

FROM GRAPH, $S_{f_a} = .91$

$F_p = S_{f_a x} W_p$ (6-6)

$= .91 \times 10.0^k$

$F_p = 9.1^k$ PER TRUSS FOR EQ-I

2. EQ-II

CONSIDER 2 CONDITIONS, MAX S_{f_a} GOVERNS

CONDITION 1

ELASTIC CAPACITY RATIO = 1.7 (GIVEN)

$S_{f_a} = 1.7 \times .91 = 1.55$

CONDITION 2

DRAW POST-YIELD RESPONSE SPECTRUM
BASED ON 10% DAMPING, 40% PERIOD INCREASE
AND 2x FIG. 2-8 VALUES
ONLY MODES 2 & 3 ARE NECESSARY TO PLOT

FIND a_{xm} FOR EACH MODE:

	1ST MODE	2ND MODE	3RD MODE	SRSS
T	1.23	.403	.23	
2x S_a	.326	.76	.76	
a_{xm}	.425	.356	.190	.586

$a_{xmII} = a_{xmI}\left(\frac{S_{a\,EQ-II}}{S_{a\,EQ-I}}\right)$ FOR MODE 1, $a_{xmII} = .360\left(\frac{.326}{.276}\right) = .425$

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Design Example F-4 4 of 6 Tank on a Building

Figure F-4. Tank on a building—continued.

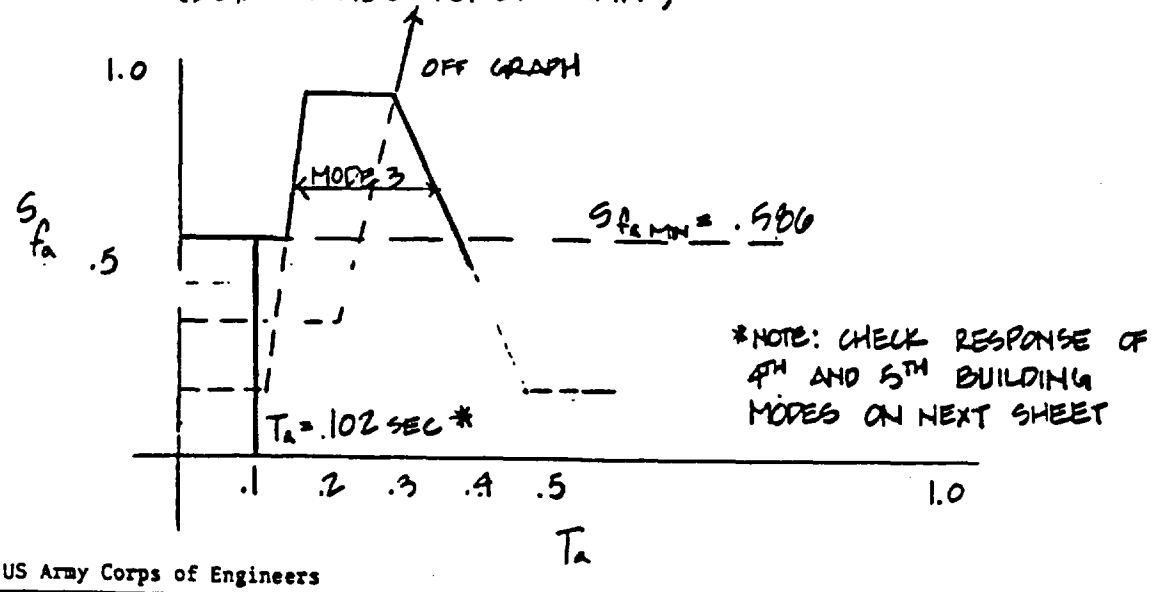
2ND MODE $T_m = .403$ $a_{xm} = .356$

T_a/T	0	.5	.7	1.5	2
M.F.	1.0	1.0	5.0	5.0	1.0
T_a	0	.202	.282	.605	.806
S_{fa}	.356	.356	1.78	1.78	.356

3RD MODE $T_m = .23$ $a_{xm} = .190$

T_a/T	0	.5	.7	1.5	2
M.F	1.0	1.0	5.0	5.0	1.0
T_a	0	.115	.161	.345	.46
S_{fa}	.190	.190	.95	.95	.190

$S_{fa\,MIN} = a_{x\,max}$ = SRSS OF a_x FOR ROOF = .586
(ESTABLISHES LOWER LIMIT)



*NOTE: CHECK RESPONSE OF 4TH AND 5TH BUILDING MODES ON NEXT SHEET

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Design Example F-4 5 of 6 Tank on a Building

Figure F-4. Tank on a building—continued.

FROM GRAPH,

$S_{fa} = .586$

SINCE $.586 < 1.55$, CONDITION 1 GOVERNS

$\therefore \quad F_p = S_{fax} W_p \qquad (6\text{-}6)$

$F_p = 1.55 \times 10^k$ PER TRUSS FOR EQ-II

NOTE FROM SHEET 5 OF 6: CHECK MODES 4 AND 5

GIVEN:

MODE 4 $T_m = 0.106$ FOR EQ-I, $PF_{xm} = 0.11$

MODE 5 $T_m = 0.073$ FOR EQ-I, $PF_{xm} = 0.05$

(SEE EXAMPLE E-1, SHEET 3 OF 7 FOR MODES 1, 2, AND 3)

REQUIRED:

S_{fa} MAXIMUM OF MODES 4 AND 5 FOR EQ-II (I.E., M.F. = 5.0)

MODE 4 $T_m = 1.4 \times 0.106 = 0.15$ SEC

$2 \times S_a$ (10% DAMPED) $= 2 \times 0.38g = 0.76g$ (FIG. 2-8)

$a_{xm} = 0.11 \times 0.76g = 0.08g$ (EQN 6-1)

$S_{fa\,MAX} = 0.08 \times 5.0 = \underline{0.40g}$ (EQN 6-4)

MODE 5 $T_m = 1.4 \times 0.073 = 0.10$ SEC, $2S_a = 0.76g$

$a_{xm} = 0.05g \times 0.76 = 0.10$ SEC, $S_{fa\,MAX} = \underline{0.20g}$

IN BOTH CASES, MAXIMUM RESPONSE IS LESS THAN SRSS OF $a_x = 0.586$.

THEREFORE, MODES 4 AND 5 DO NOT GOVERN DESIGN

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Design Example F-4	6 of 6	Tank on a Building

Figure F-4. Tank on a building—continued.

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GLOSSARY
TERMS FOR PROBABILISTIC SEISMIC RISK AND HAZARD ANALYSIS

Acceptable Risk—a probability of social or economic consequences due to earthquakes that is low enough (for example in comparison with other natural or manmade risks) to be judged by appropriate authorities to represent a realistic basis for determining design requirements for engineered structures, or for taking certain social or economic actions.

Active Fault—a fault that on the basis of historical, seismological, or geological evidence has a high probability of producing an earthquake. (Alternate: a fault that may produce an earthquake within a specified exposure time, given the assumptions adopted for a specific seismic-risk analysis.)

Attenuation Law—a description of the behavior of a characteristic of earthquake ground motion as a function of the distance from the source of energy.

B-Value—a parameter indicating the relative frequency of occurrence of earthquakes of different sizes. It is the slope of a straight line indicating absolute or relative frequency (plotted logarithmically) versus earthquake magnitude or meizoseismal Modified Mercalli intensity. (The B-value indicates the slope of the Gutenberg-Richter recurrence relationship.)

Coefficient of Variation—the ratio of standard deviation to the mean.

Damage—any economic loss or destruction caused by earthquakes.

Design Acceleration—a specification of the ground acceleration at a site, terms of a single value such as the peak or rms; used for the earthquake-resistant design of a structure (or as a base for deriving a design spectrum). See "Design Time History."

Design Earthquake—a specification of the seismic ground motion at a site; used for the earthquake-resistant design of a structure.

Design Event, Design Seismic Event—a specification of one or more earthquake source parameters, and of the location of energy release with respect to the site of interest; used for the earthquake-resistant design of a structure.

Design Ground Motion—see "Design Earthquake."

Design Spectrum—a set of curves for design purposes that gives acceleration velocity, or displacement (usually absolute acceleration, relative velocity, and relative displacement of the vibrating mass) as a function of period of vibration and damping.

Design Time History—the variation with time of ground motion (e.g., ground acceleration or velocity or displacement) at a site; used for the earthquake-resistant design of a structure. See "Design Acceleration."

Duration—a qualitative or quantitative description of the length of time during which ground motion at a site shows certain characteristics (perceptibility, violent shaking, etc.).

Earthquake—a sudden motion or vibration in the earth caused by the abrupt release of energy in the earth's lithosphere. The wave motion may range from violent at osme locations to imperceptible at others.

Elements at Risk—population, properties, economic activities, including public services etc., at risk in a given area.

Exceedence Probability—the probability that a specified level of ground motion or specified social or economic consequences of earthquakes, will be exceeded at a site or in a region during a specified exposure time.

Expected—mean, average.

Expected Ground Motion—the mean value of one or more characteristics of ground motion at a site for a single earthquake. (Mean ground motion.)

Exposure—the potential economic loss to all or certain subset of structures as a result of one or more earthquakes in an area. This term usually refers to the insured value of structures carried by one or more insurers. See "Value at Risk."

Exposure Time—the time period of interest for seismic-risk calculations, seismic-hazard calculations, or design of structures. For structures, the exposure time is often chosen to be equal to the design lifetime of the structure.

Geologic Hazard—a geologic process (e.g., landsliding, liquefaction soils, active faulting) that during an earthquake or other natural event may produce effects in structures.

Intensity—a qualitative or quantitative measure of the severity of seismic ground motion at a specific site (e.g., Modified Mercalli intensity, Rossi-Forel intensity, Housner Spectral intensity, Arias intensity, peak acceleration, etc.).

Loss—any adverse economic or social consequence cause by one or more earthquakes.

Maximum—the largest value attained by a variable during a specified exposure time. See "Peak Value."

Maximum Credible
Maximum Expectable
Maximum Expected
Maximum Probable—These terms are used to specify the largest value of a variable, for example, the magnitude of an earthquake, that might reasonably be expected to occur. *These are misleading terms and their use is discouraged.* (The U.S. Geological Survey and some individuals and companies define the maximum credible earthquake as "the largest earthquake that can be reasonably expected to occur." The Bureau of Reclamation, the First Interagency Working Group (Sept. 1978) defined the maximum credible earthquake as "the earthquake that would cause the most severe vibratory ground motion capable of being produced at the site under the current known tectonic framework." It is an event that can be supported by all known geologic and seismologic data. The maximum expectable or expected earthquake is defined by USGS as "the largest earthquake that can be reasonably expected to occur." The maximum probable earthquake is sometimes defined as the worst historic earthquake. Alternatively, it is defined as the 100-year-return-period earthquake, or an earthquake that probabilistic determination of recurrence will take place during the life of the structure.)

Maximum Possible—the largest value possible for a variable. This follows from an explicit assumption that larger values are not possible, or implicitly from assumptions that related variables or functions are limited in range. The maximum possible value may be expressed deterministically or probabilistically.

Mean Recurrence Interval, Average Recurrence Interval—the average time between earthquakes or faulting vents with specific characteristics (e.g., magnitude ≥ 6) in a specified region or in a specified fault zone.

Mean Return Period—the average time between occurrences of ground motion with specific characteristics (e.g., peak horizontal acceleration ≥ 0.1 g) at a site. (Equal to the inverse of the annual probability of exceedance.)

Mean Square—expected value of the square of the random variable. (Mean square minus square of the mean gives the variance of random variable.)

Peak Value—the largest value of a time-dependent variable during an earthquake.

Response Spectrum—a set of curves calculated from an earthquake accelerogram that gives values of peak response of a damped linear oscillator, as a function of its period of vibration and damping.

Root Mean Square (rms)—square root of the mean square value of a random variable.

Seismic-Activity Rate—the mean number per unit time of earthquakes with specific characteristics (e.g., magnitude ≥ 6) originating on a selected fault or in a selected area.

Seismic-Design-Load Effects—the actions (axial forces, shears, or bending moments) and deformations induced in a structural system due to a specified representation (time history, response spectrum, or base shear) or seismic design ground motion.

Seismic-Design Loading—the prescribed representation (time history, response spectrum, or equivalent static base shear) of seismic ground motion to be used for the design of a structure.

Seismic Event—the abrupt release of energy in the earth's lithosphere, causing an earthquake.

Seismic Hazard—any physical phenomenon (e.g., ground shaking, ground failure) associated with an earthquake that may produce adverse effects on human activities.

Seismic Risk—the probability that social or economic consequences of earthquakes will equal or exceed specified values at a site, at several sites, or in an area, during a specified exposure time.

Seismic-Risk Zone—an obsolete term. See "Seismic Zone."

Seismic-Source Zone—an obsolete term. See "Seismogenic Zone" and "Seismotectonic Zone."

Seismic Zone—a generally large area within which seismic-design requirements for structures are constant.

Seismic Zoning, Seismic Zonation—the process of determining seismic hazard at many sites for the purpose of delineating seismic zones.

Seismic Microzone—a generally small area within which seismic-design requirements for structures are uniform. Seismic microzones may show relative ground motion amplification due to local soil conditions without specifying the absolute levels of motion or seismic hazard.

Seismic Microzoning, Seismic Microzonation—the process of determining absolute or relative seismic hazard at many sites, accounting for the effects of geologic and topographic amplification of motion and of soil stability and liquefaction, for the purpose of delineating seismic microzones. Alternatively, microzonation is a process for identifying detailed geological, seismological, hydrological, and geotechnical site characteristics in a specific region and incorporating them into land-use planning and the design of safe structures in order to reduce damage to human life and property resulting from earthquakes.

Seismogenic Zone, Seismogenic Province—a planar representation of a three-dimensional domain in the earth's lithosphere in which earthquakes are inferred to be of similar tectonic origin. A seismogenic zone may represent a fault in the earth's lithosphere. See "Seismotectonic Zone."

Seismogenic Zoning—the process of delineating regions have nearly homogeneous tectonic and geologic character, for the purpose of drawing seismogenic zones. The specific procedures used depend on the assumptions and mathematical models used in the seismic-risk analysis or seismic-hazard analysis.

Seismotectonic Zone, Seismotectonic Providence—a seismogenic zone in which the tectonic processes causing earthquakes have been identified. These zones are usually fault zones.

Source Variable—a variable that describes a physical characteristic (e.g., magnitude, stress drop, seismic moment, displacement) of the source of energy release causing an earthquake.

Standard Deviation—the square root of the variance of a random variable.

Upper Bound—see "Maximum Possible."

Value at Risk—the potential economic loss (whether insured or not) to all or certain subset of structures as a result of one or more earthquakes in an area. See "Exposure."

Variance—the mean squared deviation of a random variable from its average value.

Vulnerability—the degree of loss to a given element at risk, or set of such elements, resulting from an earthquake of a given magnitude or intensity, which is usually expressed on a scale from 0 (no damage) to 10 (total loss).

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