

TECHNICAL MANUAL

**SEISMIC DESIGN GUIDELINES FOR
ESSENTIAL BUILDINGS**



HEADQUARTERS, DEPARTMENT OF THE ARMY
27 FEBRUARY 1986

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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 3-5a.

(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 3-5b.

(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 characteristics, 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 pe-

riods 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.

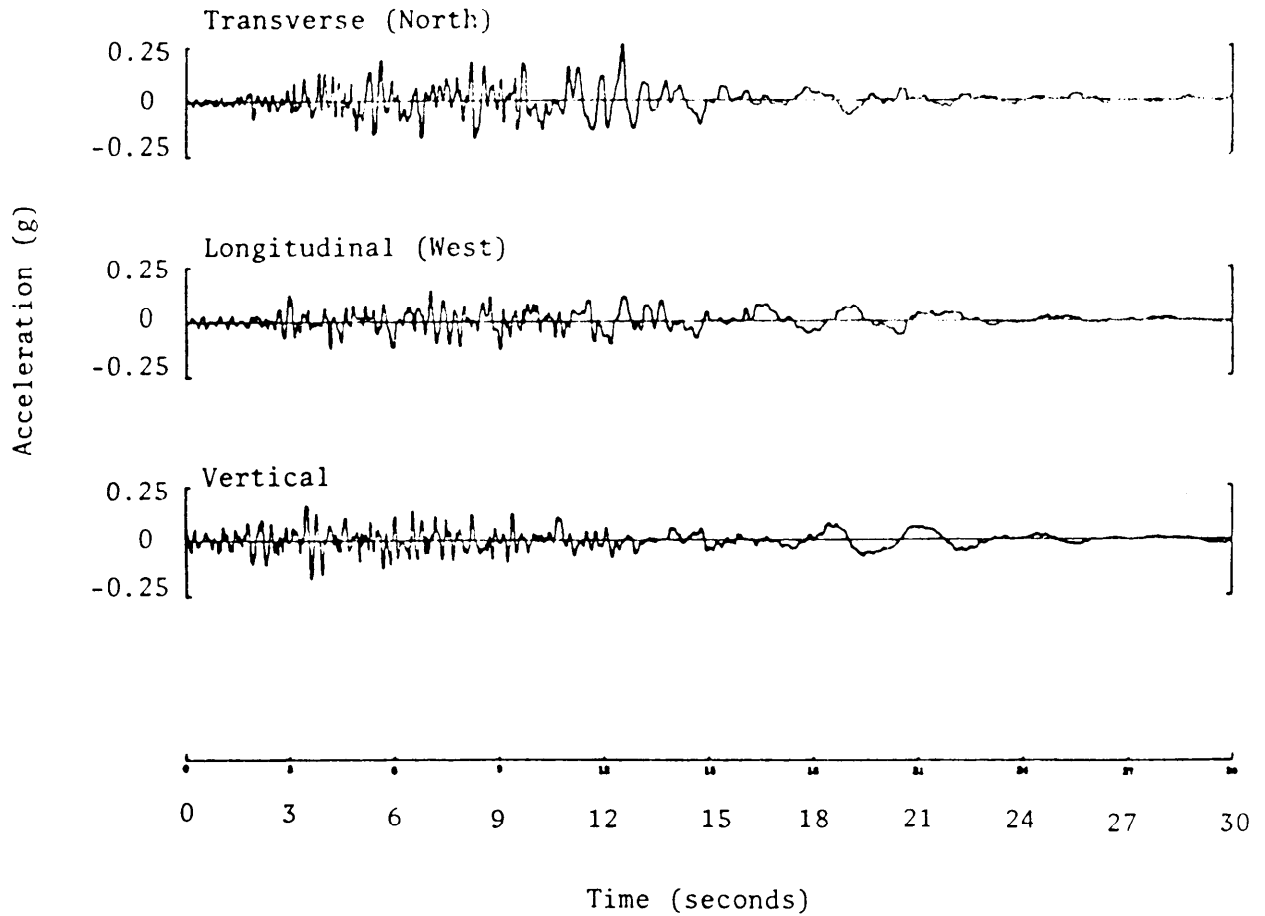
(2) The predominant periods of ground motion vibration generally lengthen as distance increases from the source (para 3-6f).

(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.

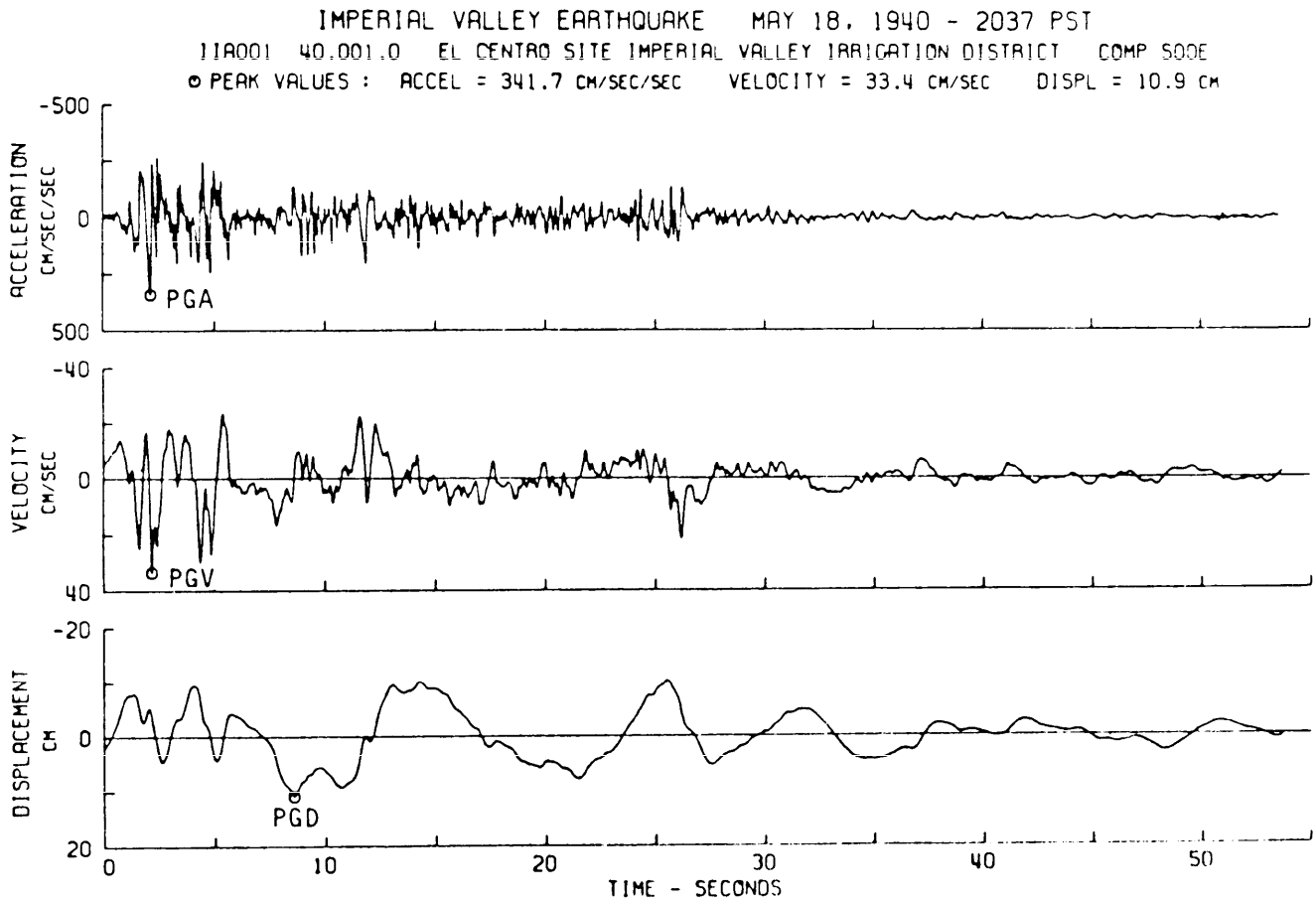
b. Time history. The basic measurement of earthquake ground motion is the accelerogram record taken by seismometers. When these instrument records are properly corrected for elimination of recording noise and for base line adjustment, a primary data base for seismic load specifications is provided. Data 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 $\dot{x}(t)$; 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.

RECORDED ACCELERATION AT GROUND LEVEL DURING THE 1971
SAN FERNANDO EARTHQUAKE



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.

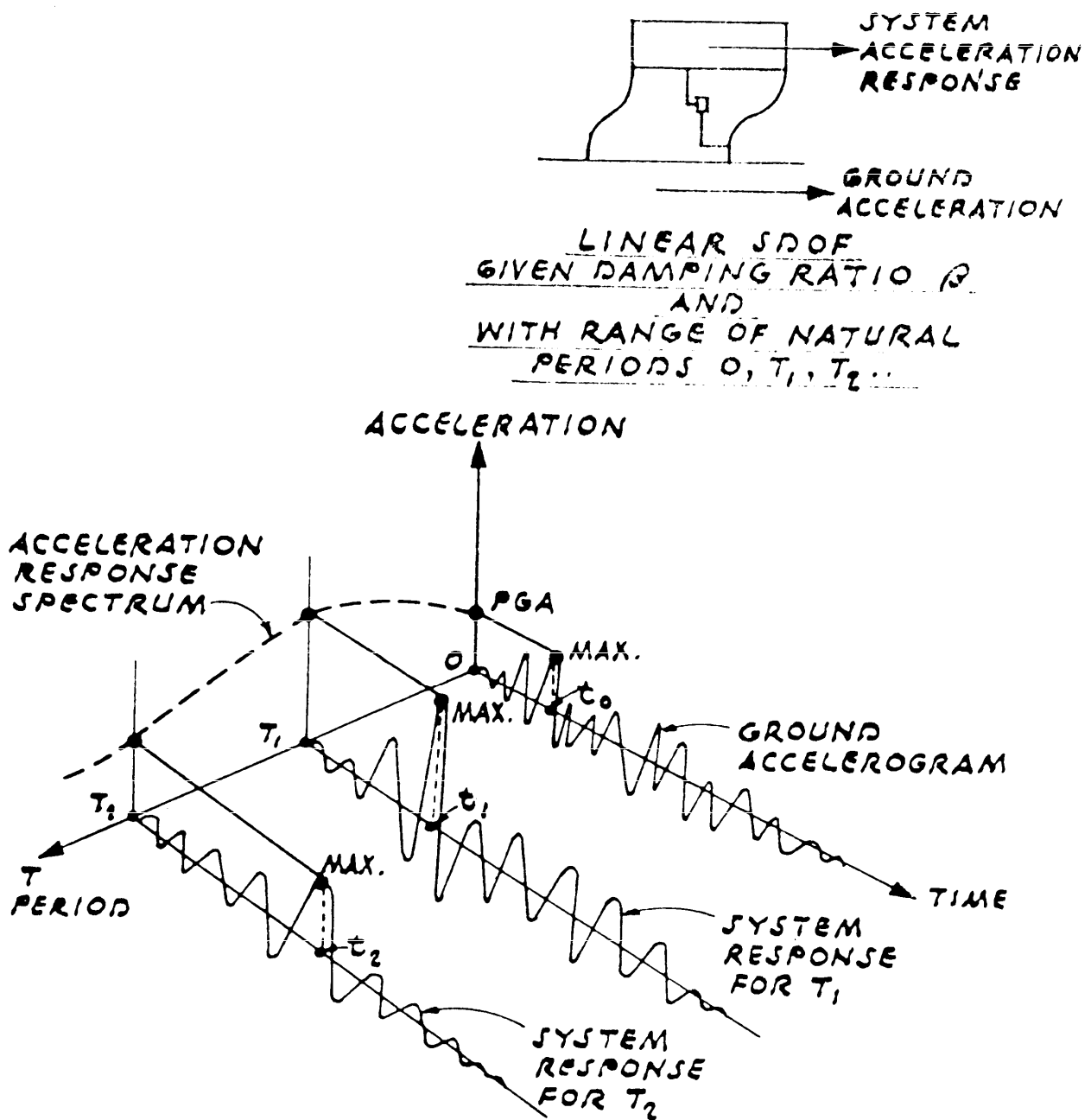


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 U.S. Coast and Geodetic Surveys, U.S. Govern-
 ment Printing Office, 1940.

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 0, 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



US Army Corps of Engineers

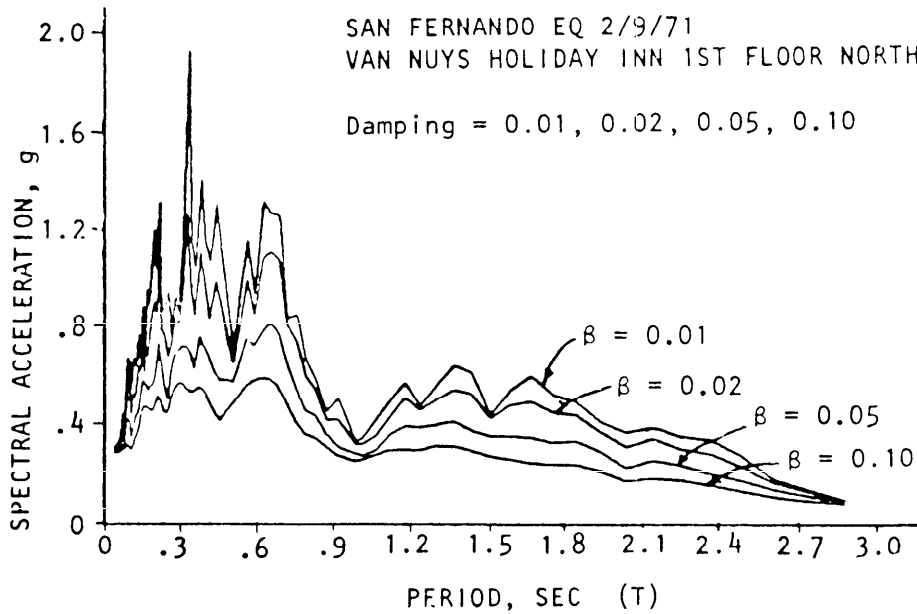
Figure 2-3. Description of acceleration response spectrum.

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 2-4b), or averages of multiple record spectra, is employed as the seismic load input for the dynamic analysis of structures. Note that the S_a

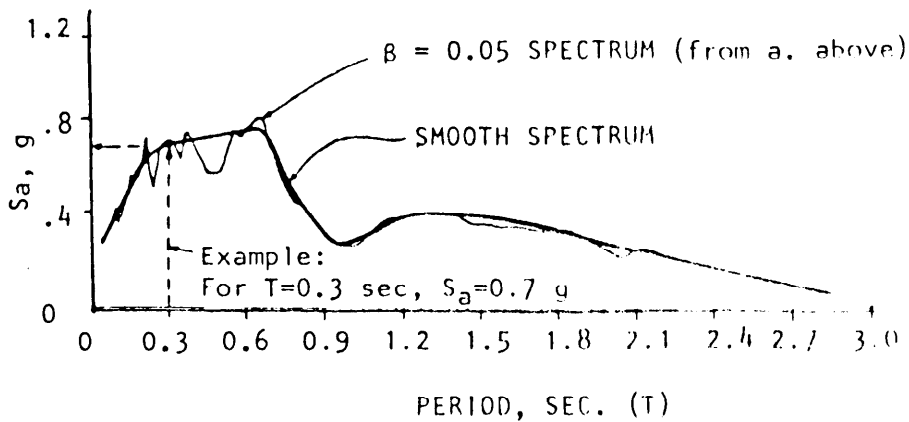
curve provides the maximum response value for any given system period, T .

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: $\beta = 0.05$

US Army Corps of Engineers

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.

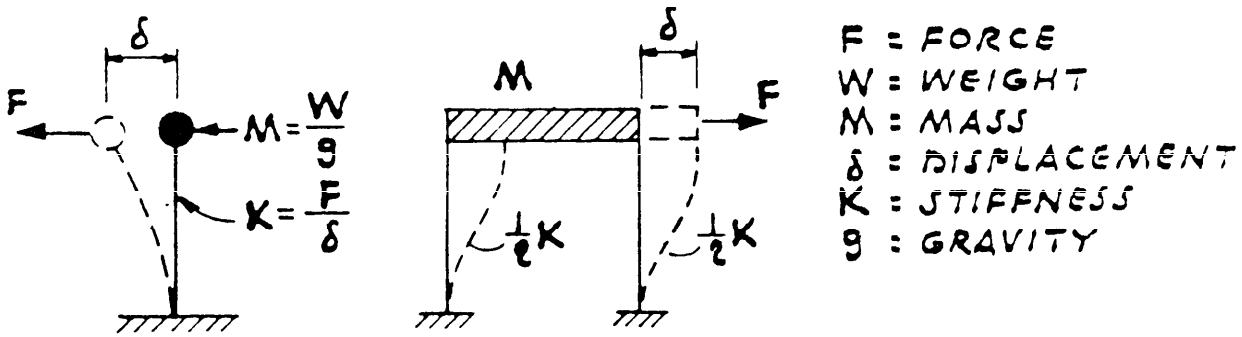
2-5. Dynamic analysis of structures.

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 2-3c).

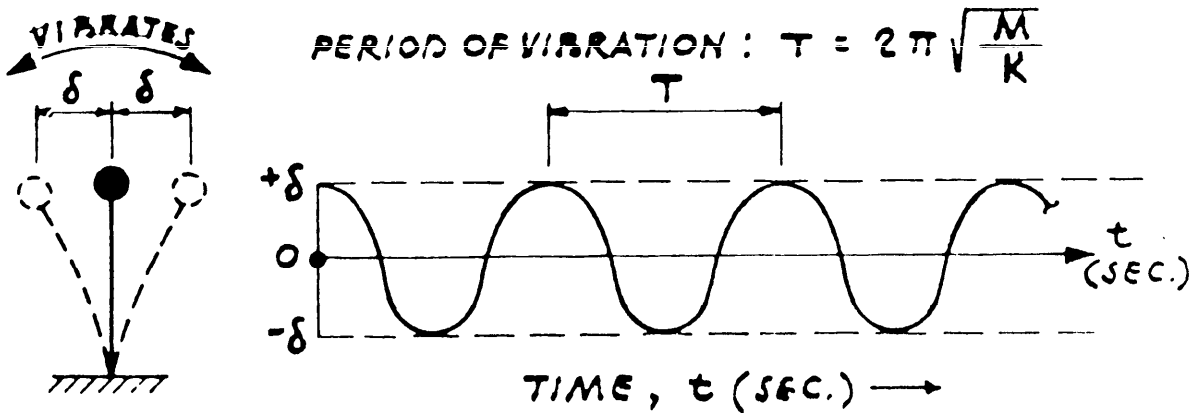
a. Single-degree-of-freedom system. One fundamental system that is investigated by dynamic analysis is the simple oscillator or SDOF system, as shown in figure 2-5. Represented by a single lump of mass on the upper end of a vertically cantilevered pole or by a mass supported by two columns (part a of 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 frequency 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.

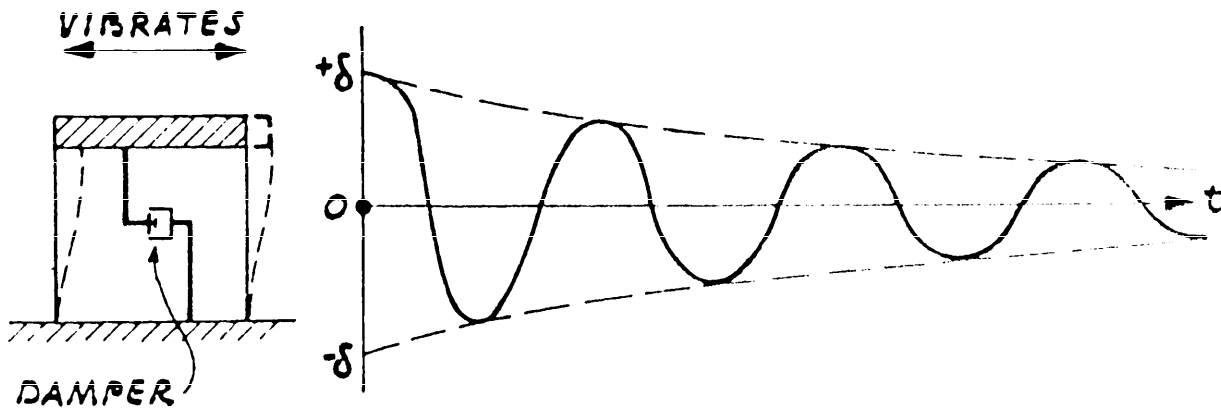
b. Multi-degree-of-freedom systems. Multi-story buildings are analyzed as MDOF systems as shown in figure 2-6. They can be represented by lumped masses attached at intervals along the length of a vertically cantilevered pole (part a of fig 2-6). Each mass can be deflected in one direction or another; 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-



a. IDEALIZED SINGLE LUMPED-MASS SYSTEMS

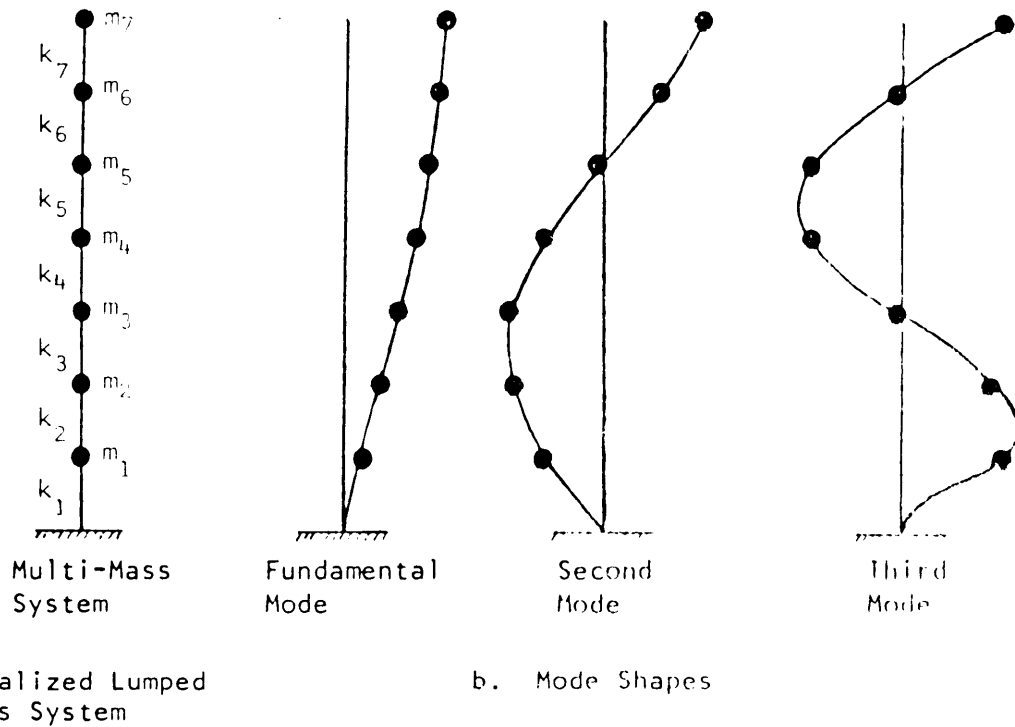


b. FREE VIBRATION (NO DAMPING)



c. DAMPED FREE VIBRATION (DAMPED AT A RATIO TO CRITICAL DAMPING EQUAL TO β)

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



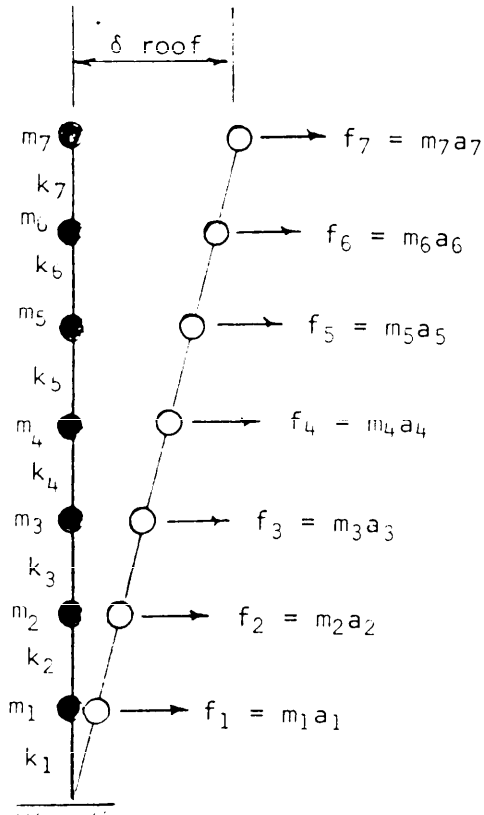
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Figure 2-6. Multi-degree-of-freedom system.

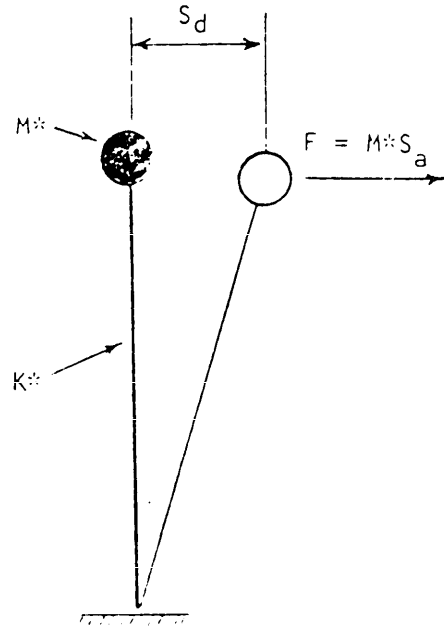
ural modal period of vibration with a unique mode shape being formed by a line connecting the deflected masses (part b of fig 2-6). When oscillating motion is applied to the base of the multi-mass system, these masses move. The deflected shape is a combination of all the mode shapes; but modes having 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

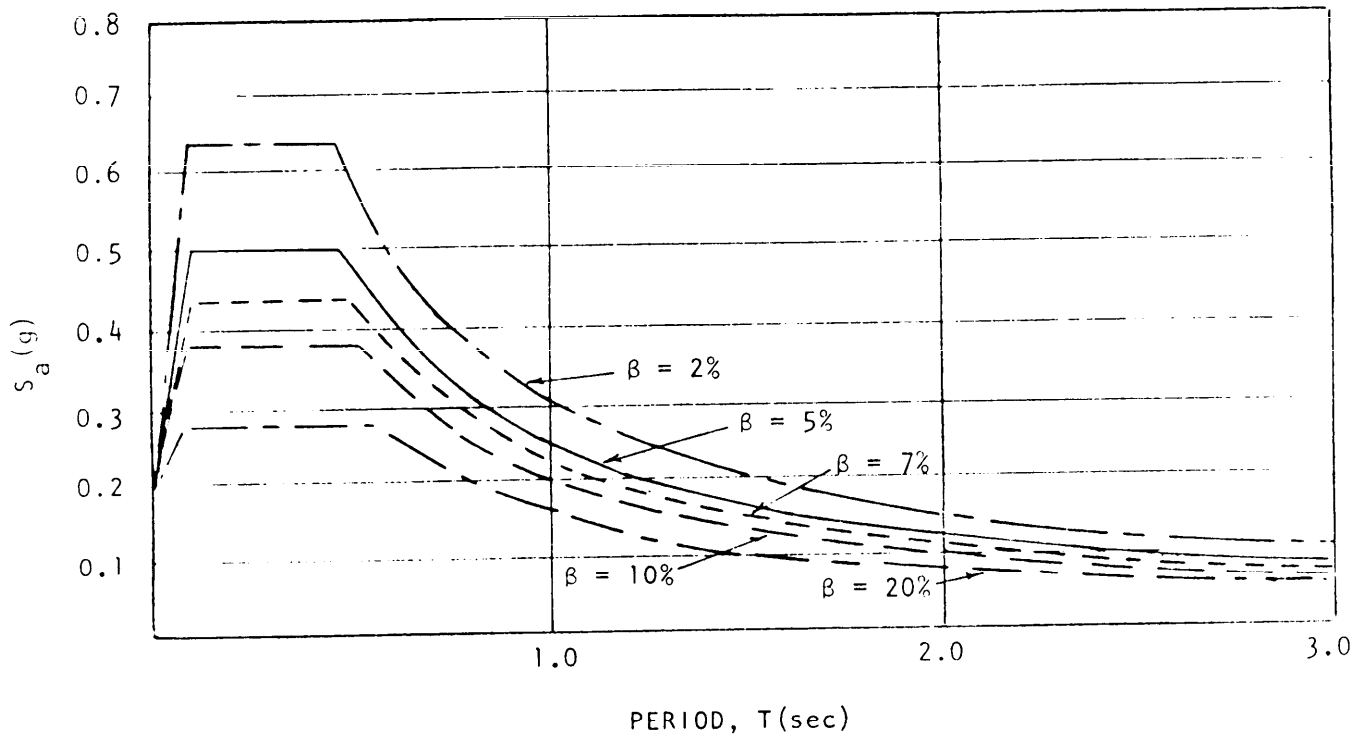
M^* and K^* are normalized values of mass and stiffness that represent the equivalent combined effects of the story masses (M) and stiffnesses (K)

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

illustrate 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.



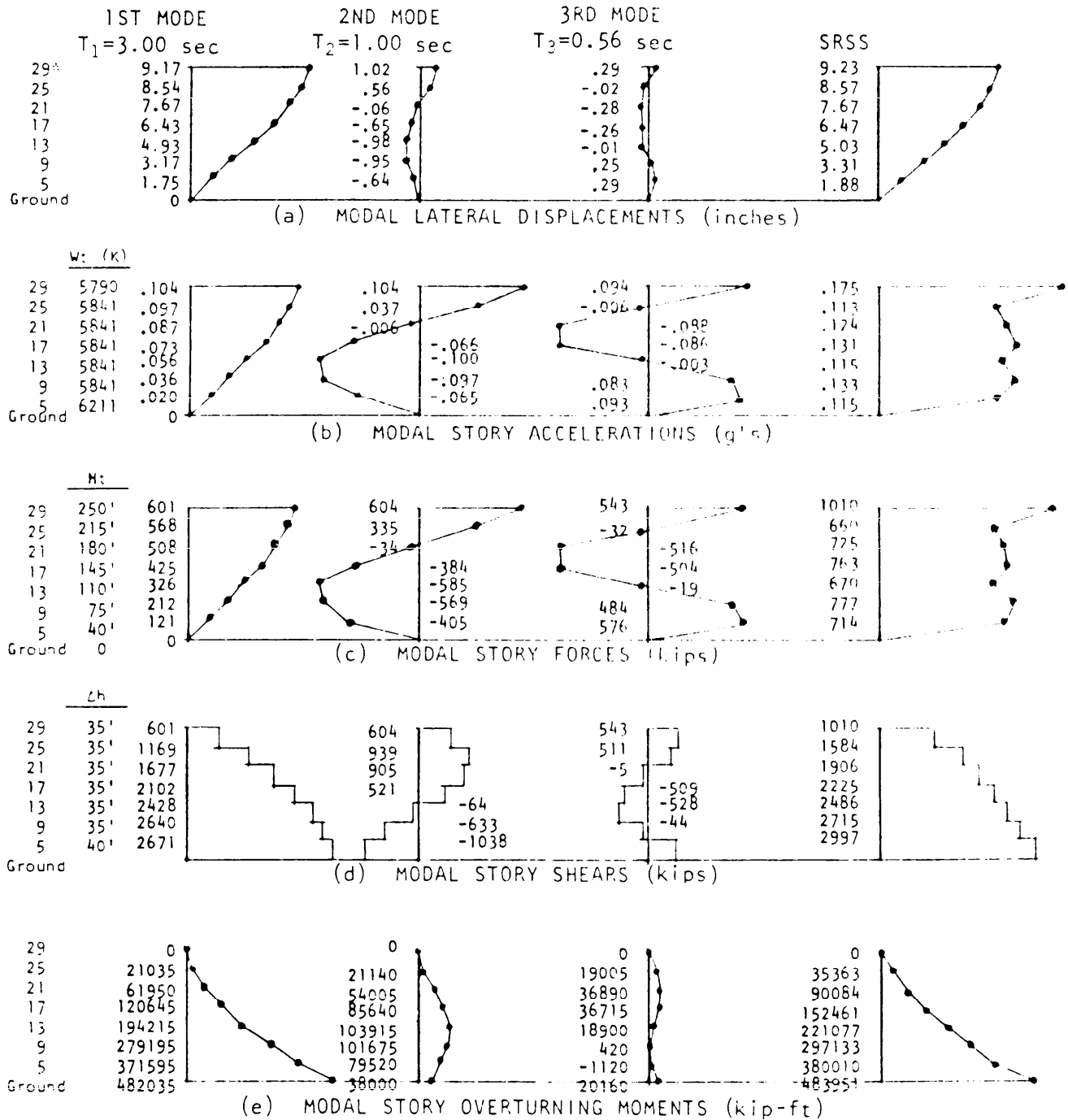
SPECTRAL ACCELERATION, S_a (g)												
$T \backslash \beta$	0.1	0.48	0.50	0.80	1.0	1.25	1.5	1.75	2.0	2.25	2.5	3.0
2%	0.64	0.64	0.59	0.37	0.30	0.24	0.20	0.17	0.15	0.13	0.12	0.10
5%	0.50	0.50	0.48	0.30	0.24	0.192	0.16	0.137	0.12	0.107	0.096	0.08
7%	0.44	0.44	0.44	0.28	0.22	0.18	0.15	0.13	0.11	0.10	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 ($\beta = 0.05$), 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.

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

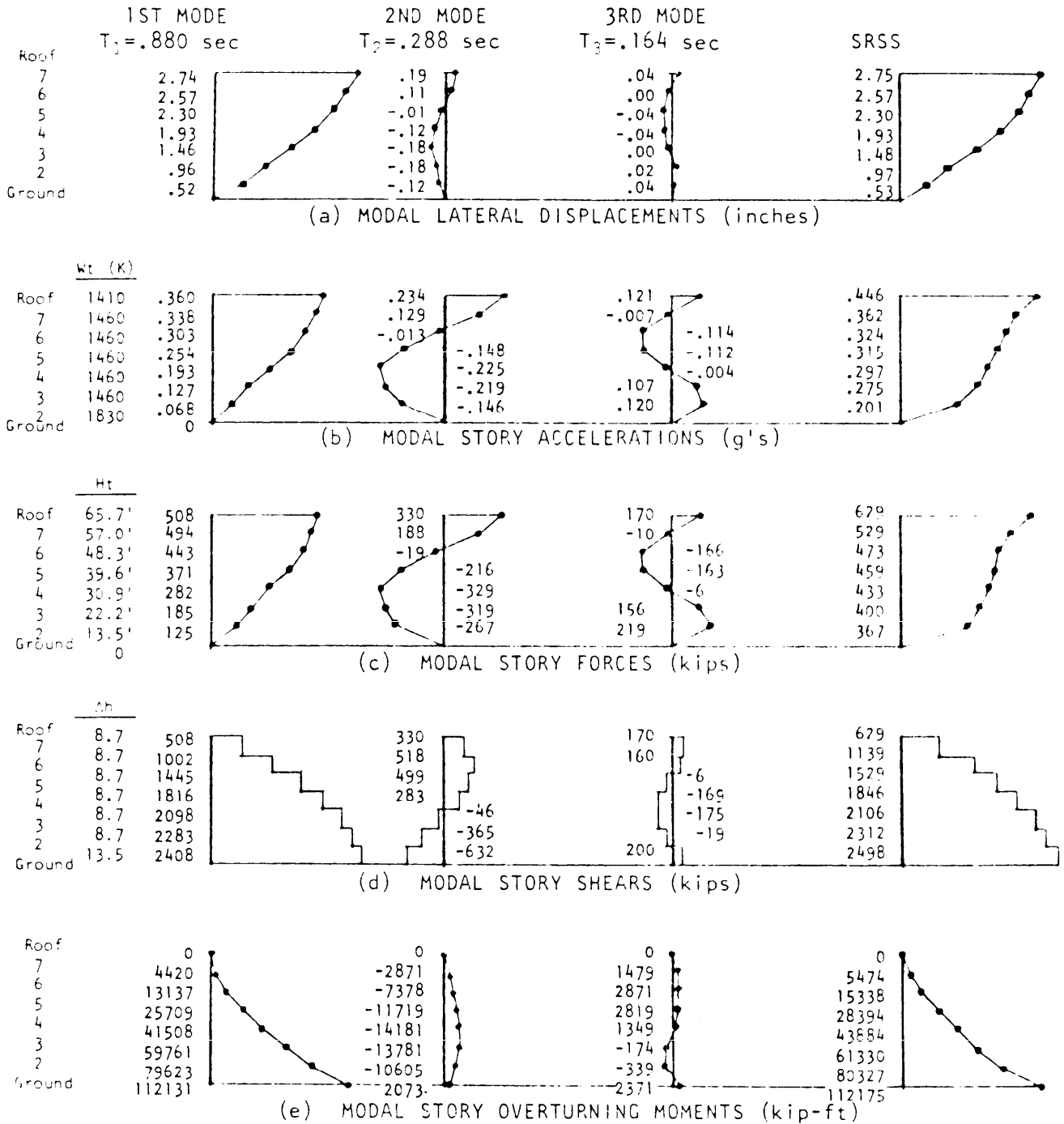
higher modes become somewhat insignificant because of the reversal of force directions. The SRSS curve is essentially equal to the first mode curve at the lower stories of the building.

(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 ($\beta = 0.05$) 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.

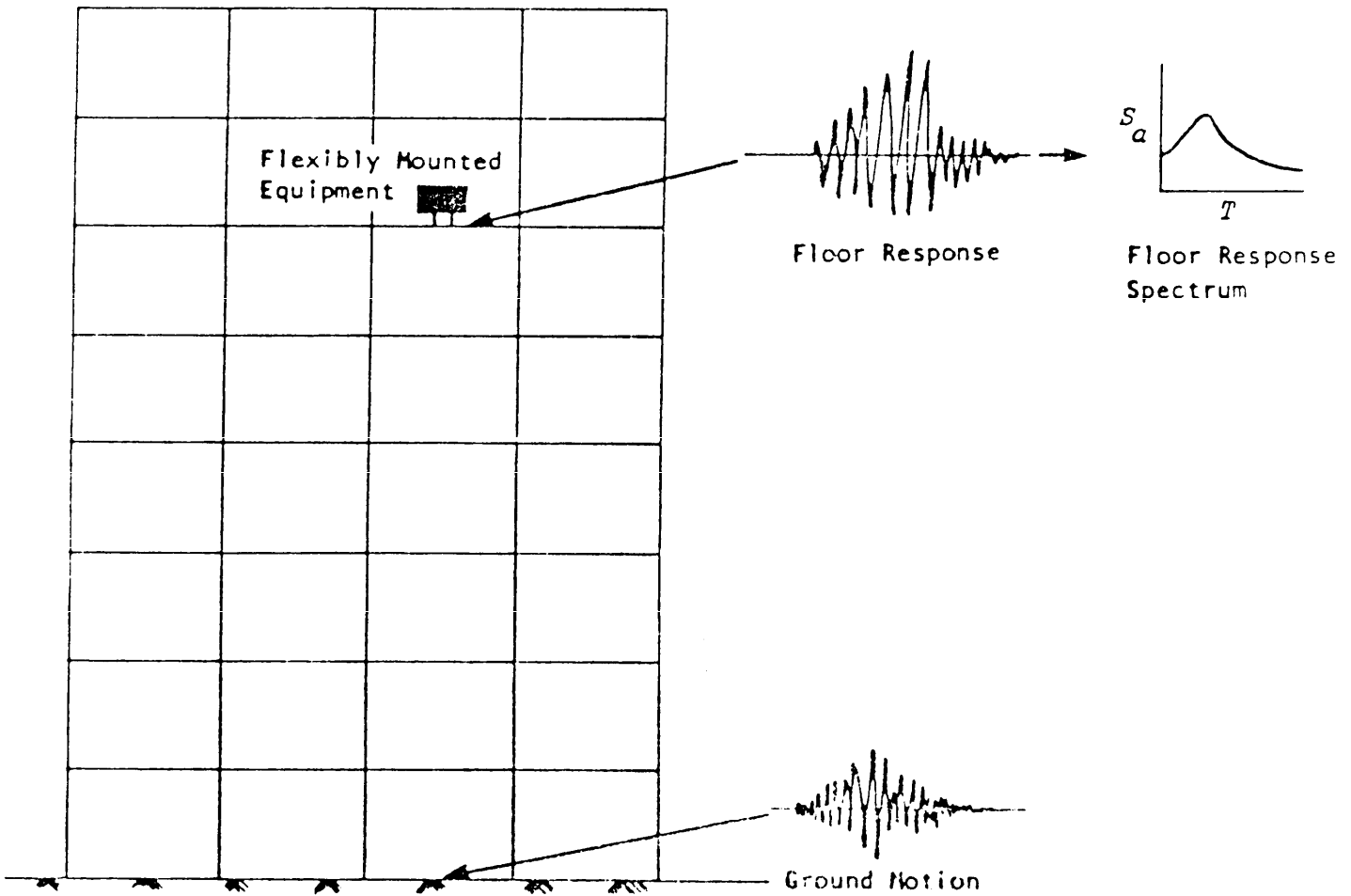
2-6. Nonstructural elements.

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 3-4a, 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 3-1b(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 3-6c(3) or 3-6f(3) as directed by the responsible agency. If this WUS site location is outside of the limits of paragraph 3-1b(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 3-1b(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	II
	20 Kilometers or less*	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	II
	Within a province having $M_{max} \geq 6.0$	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

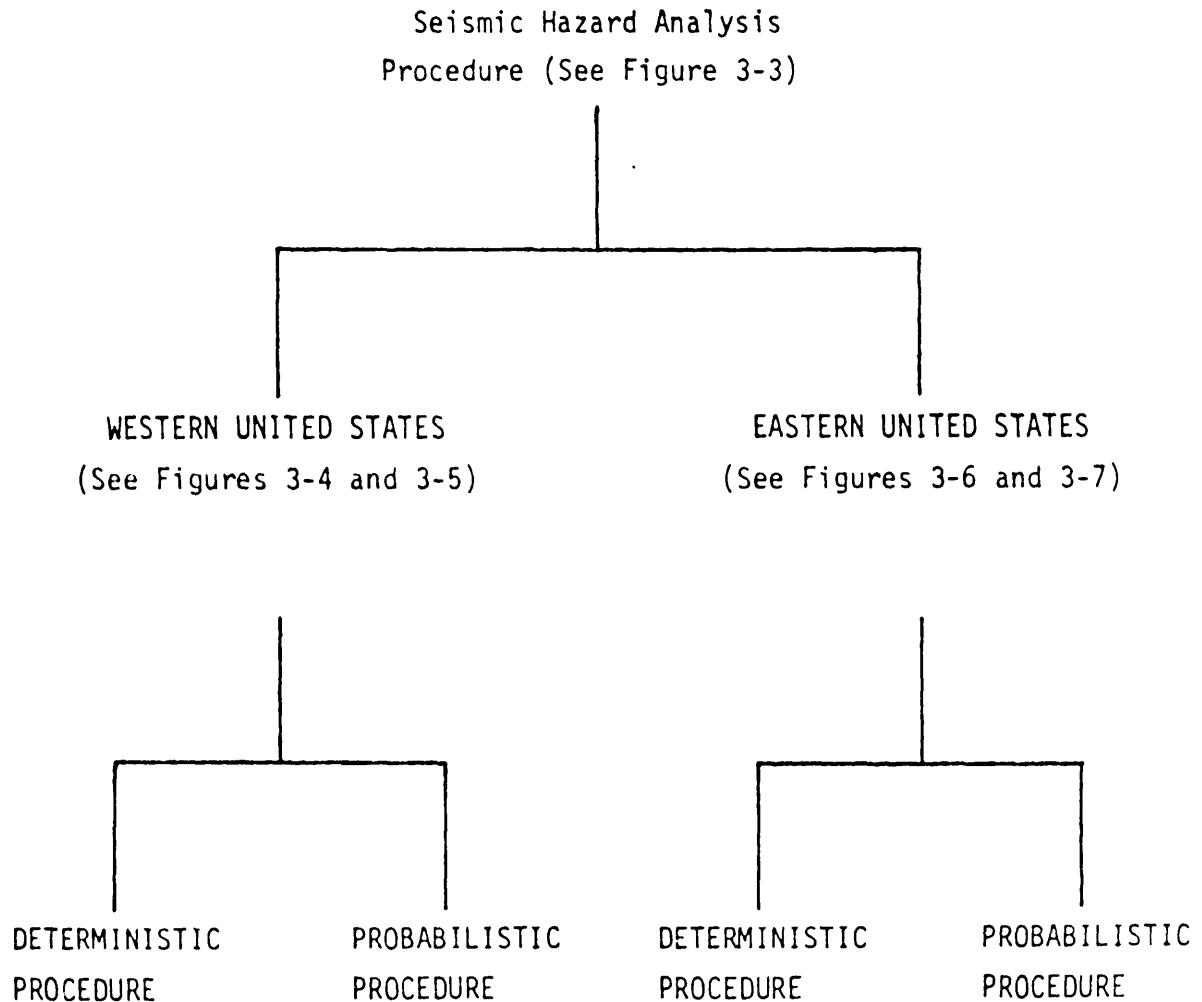
Table 3-1. Return period as a function of exposure time and probability of non-exceedance

Exposure Time Years	10	20	30	40	50	100
"Hazard" or Probability of exceeding %						
5	195	390	585	780	975	1950
10	95	190	285	390	475	<u>950</u>
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	<u>72</u>	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).

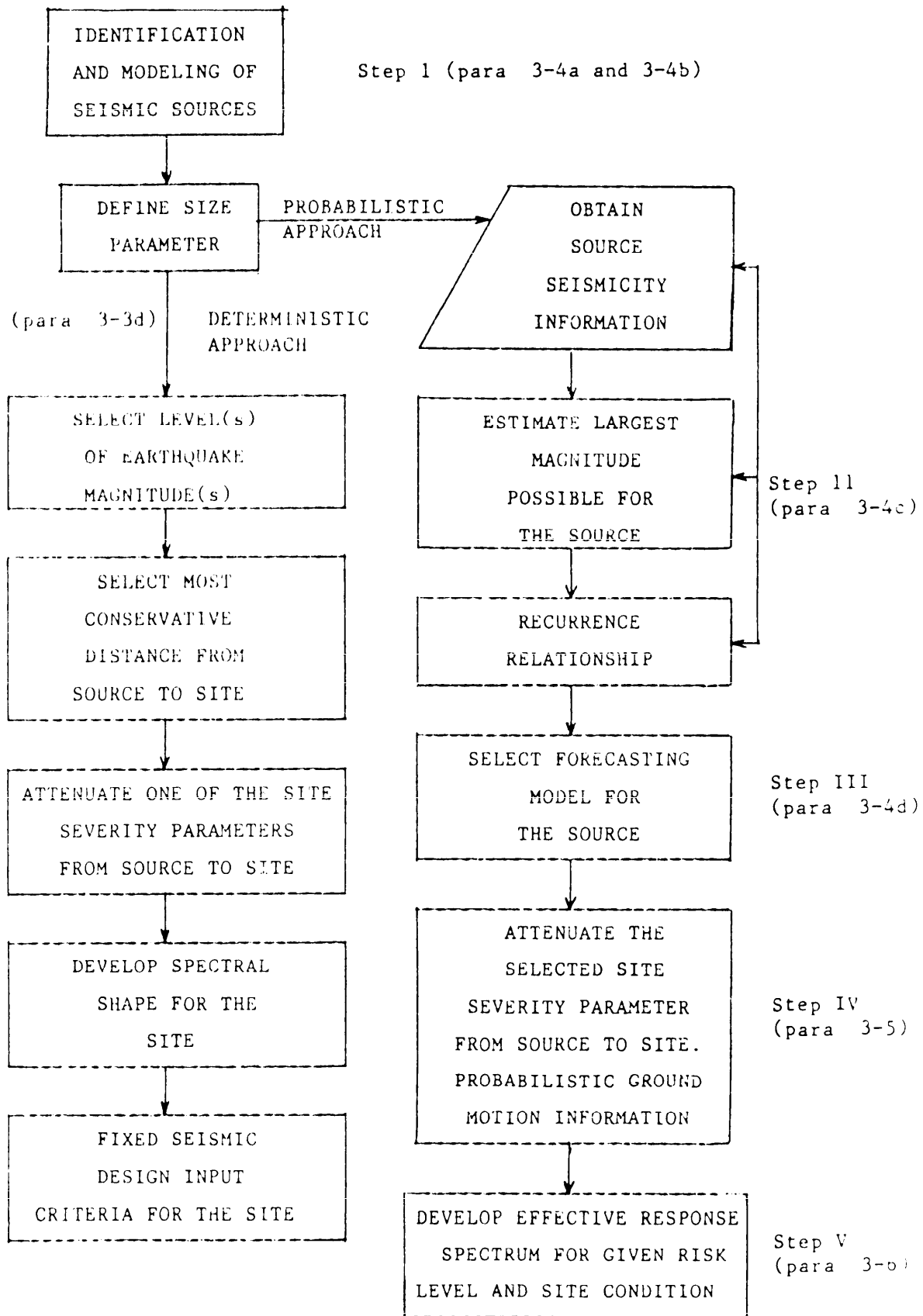
parameters (such as magnitude, intensity, and spectra) to be employed are dependent upon the type of information available to the analyst and the needs of the designer. The procedures and the models selected depend on the type, quantity, and quality of information as well as the goal of the analysis. 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-

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



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Figure 3-2. General flow diagram selection chart.



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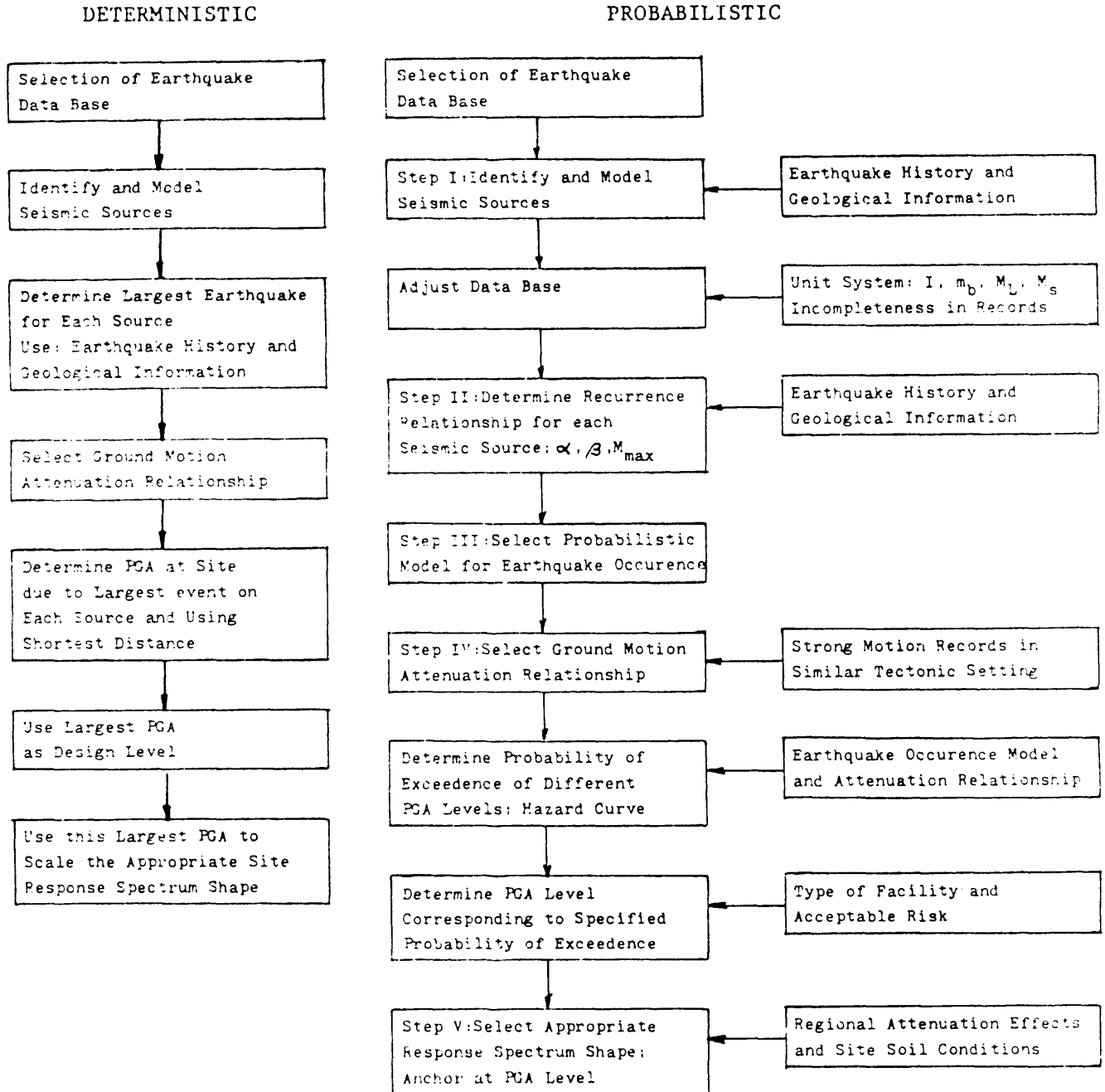
Figure 3-3. General flow chart.

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 in-

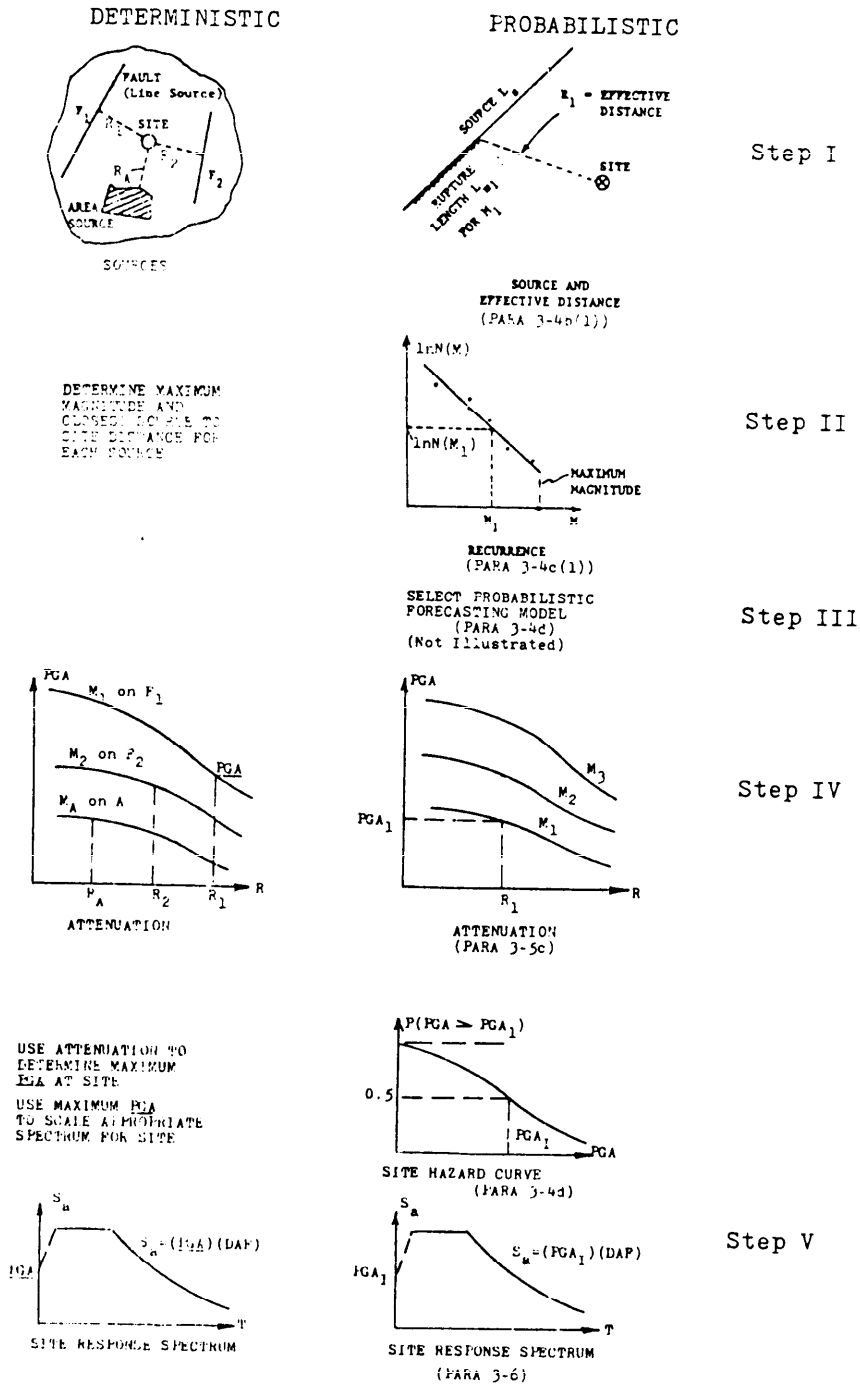
formation from experts. The purpose of this step is to assemble the information required to delineate faults and regions within which seismic activity can be considered homogeneous. See paragraph 3-4b for a detailed discussion and appendix D for examples.

(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



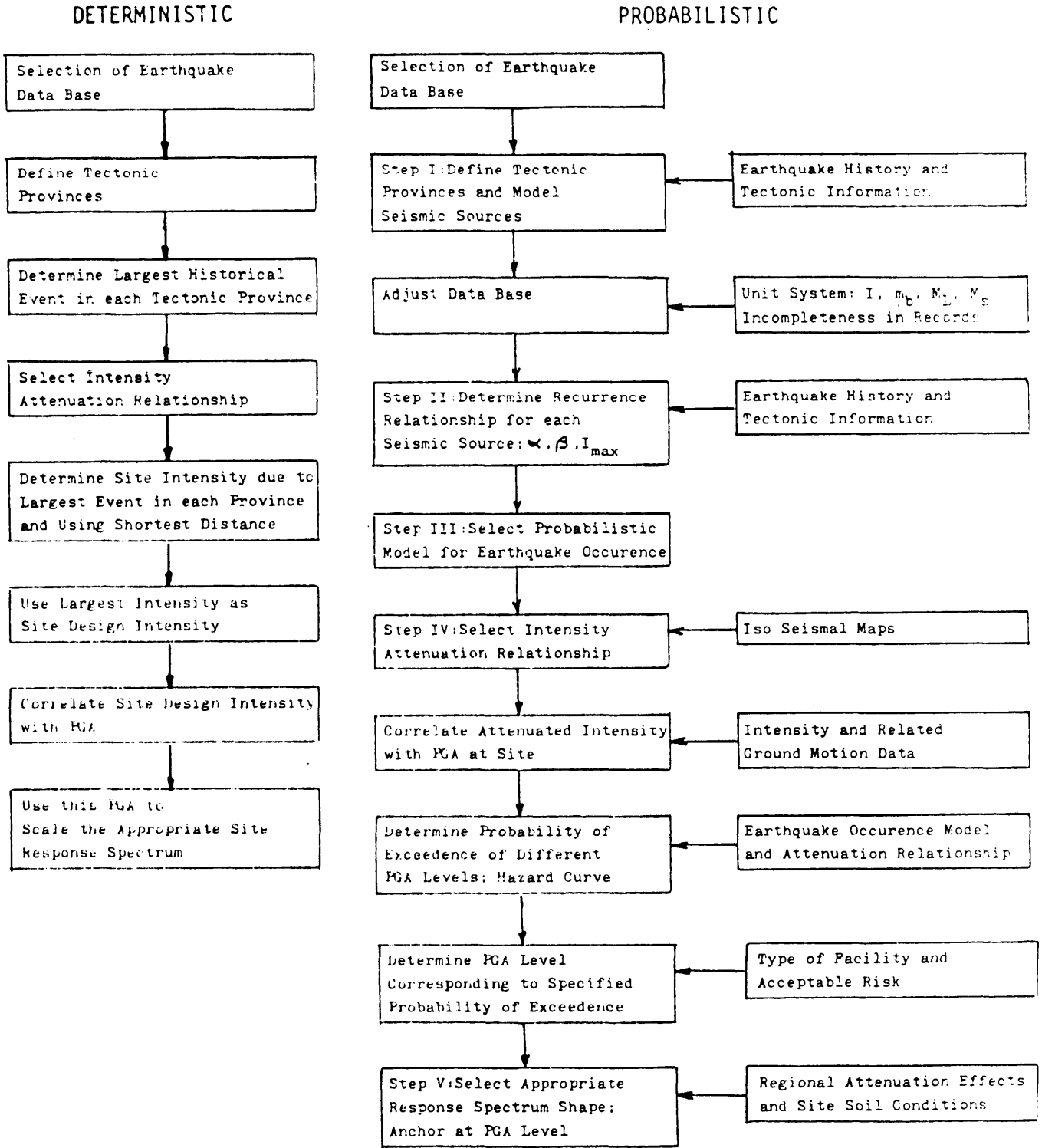
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Figure 3-4. Flow diagram for the Western United States.



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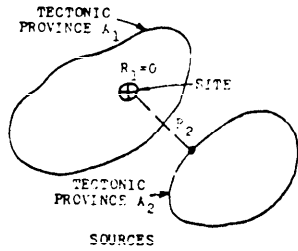
Figure 3-5. Hazard evaluation of WUS.



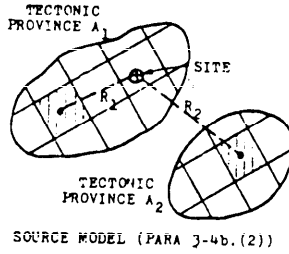
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Figure 3-6. Flow diagram for the Eastern United States.

DETERMINISTIC

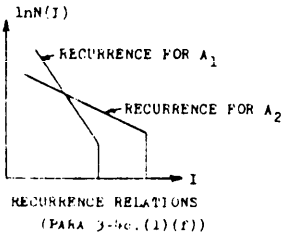


PROBABILISTIC



Step I

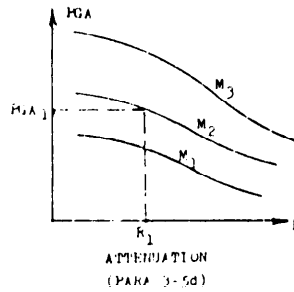
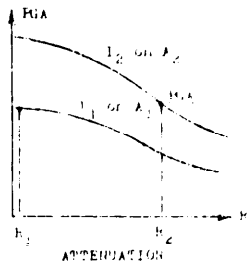
DETERMINE MAXIMUM INTENSITY AND CLOSEST SOURCE TO SITE DISTANCE FOR EACH SOURCE



Step II

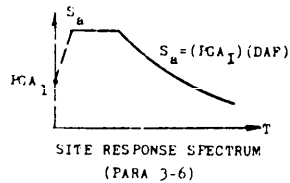
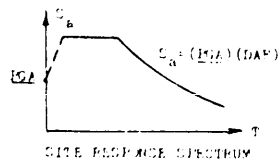
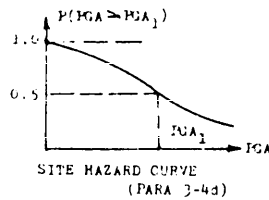
SELECT PROBABILISTIC FORECASTING MODEL (PARA 3-4d) (Not Illustrated)

Step III



Step IV

USE ATTENUATION TO DETERMINE MAXIMUM MGA AT SITE
USE MAXIMUM MGA TO SCALE APPROPRIATE SPECTRUM FOR SITE



Step V

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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 exceedance of that magnitude anywhere on the source under consideration. The upper magnitude cut-off 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 3-4c 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 3-4d 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 ac-

celeration 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 3-8g 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 con-

tained 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 3-4c.
- 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 occurrence 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 occurrence rates.
- Low attenuation of ground motion severity with significant values at 200 to 300 kilometers.

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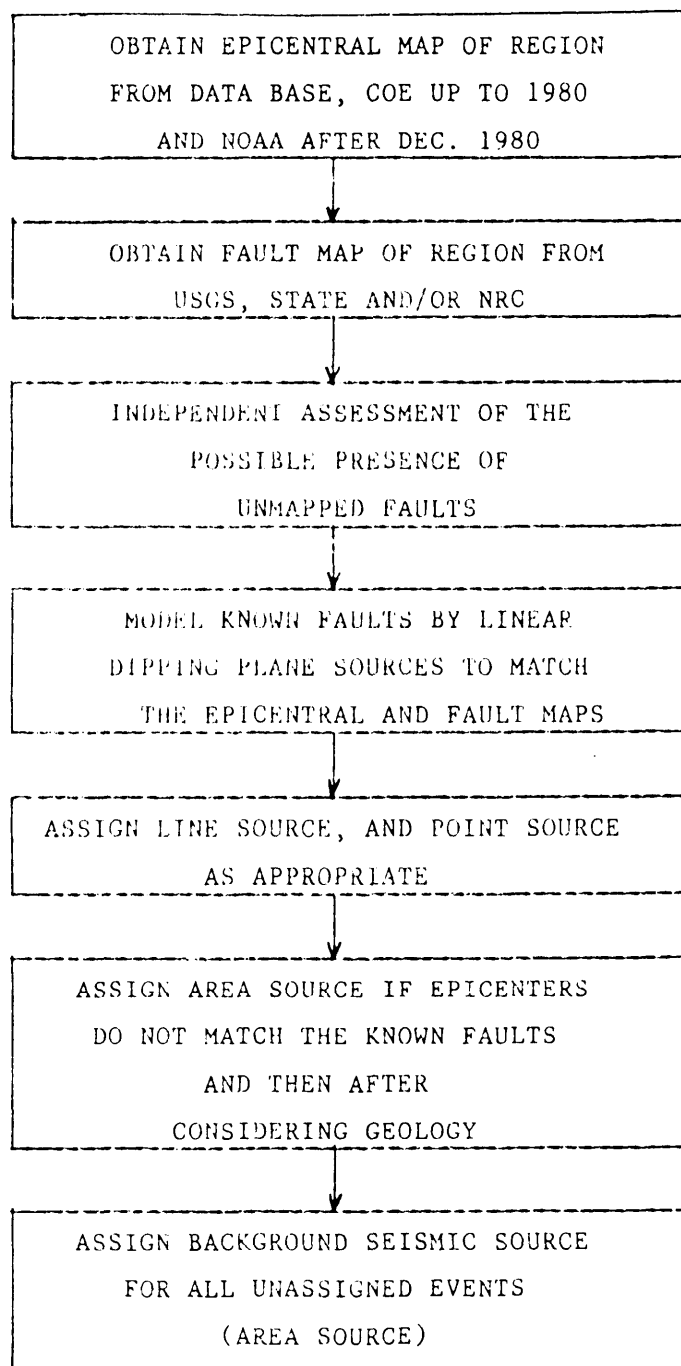
Figure 3-8. Regional differences.

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

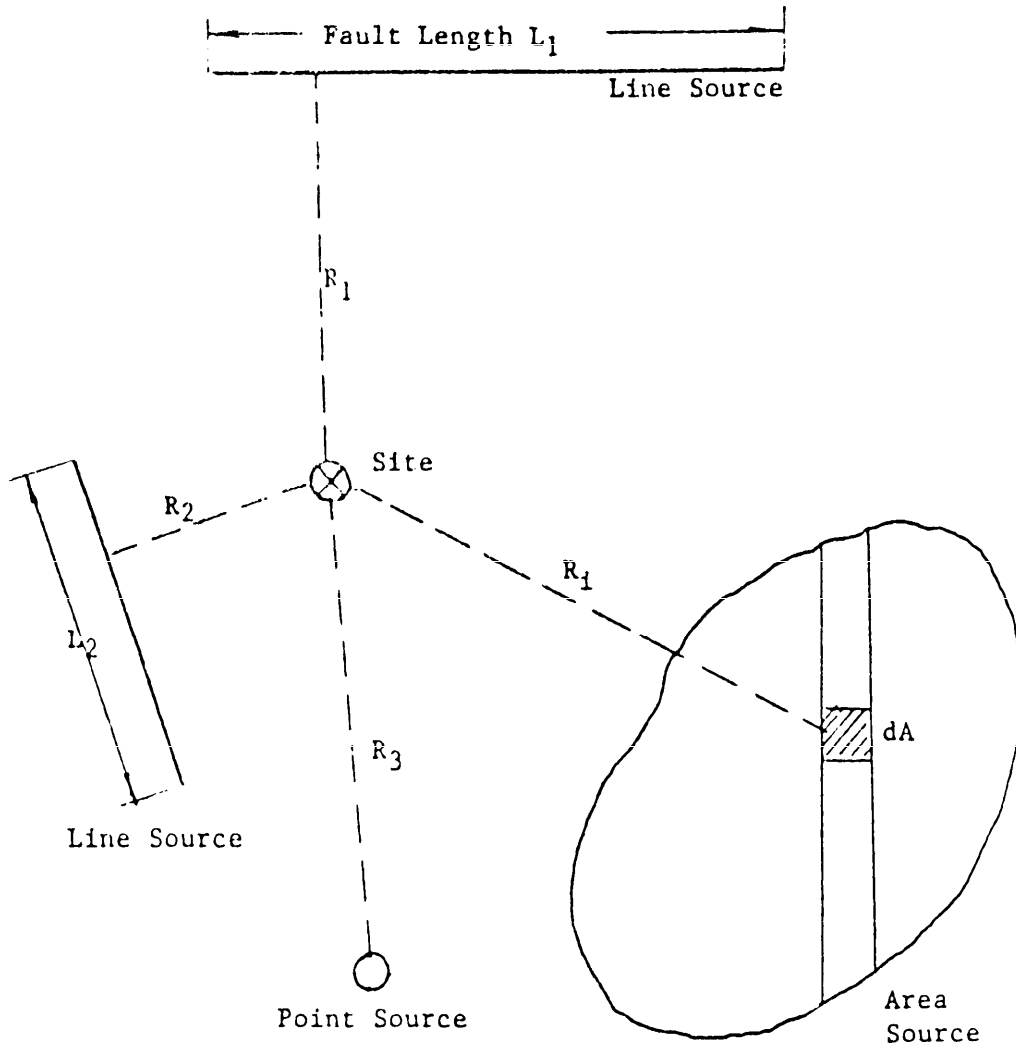
planes and line sources are used: (Biblio 41), and (Biblio 70). An example to demonstrate as to how sources are modelled is given in appendix D.

(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.

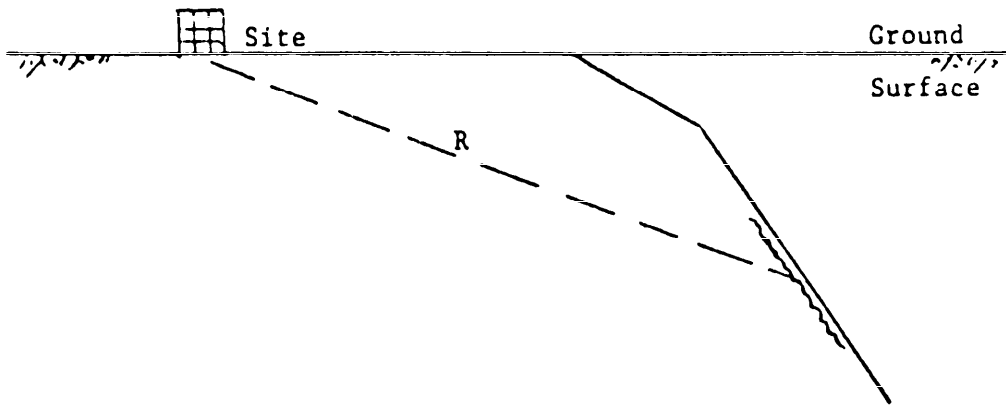
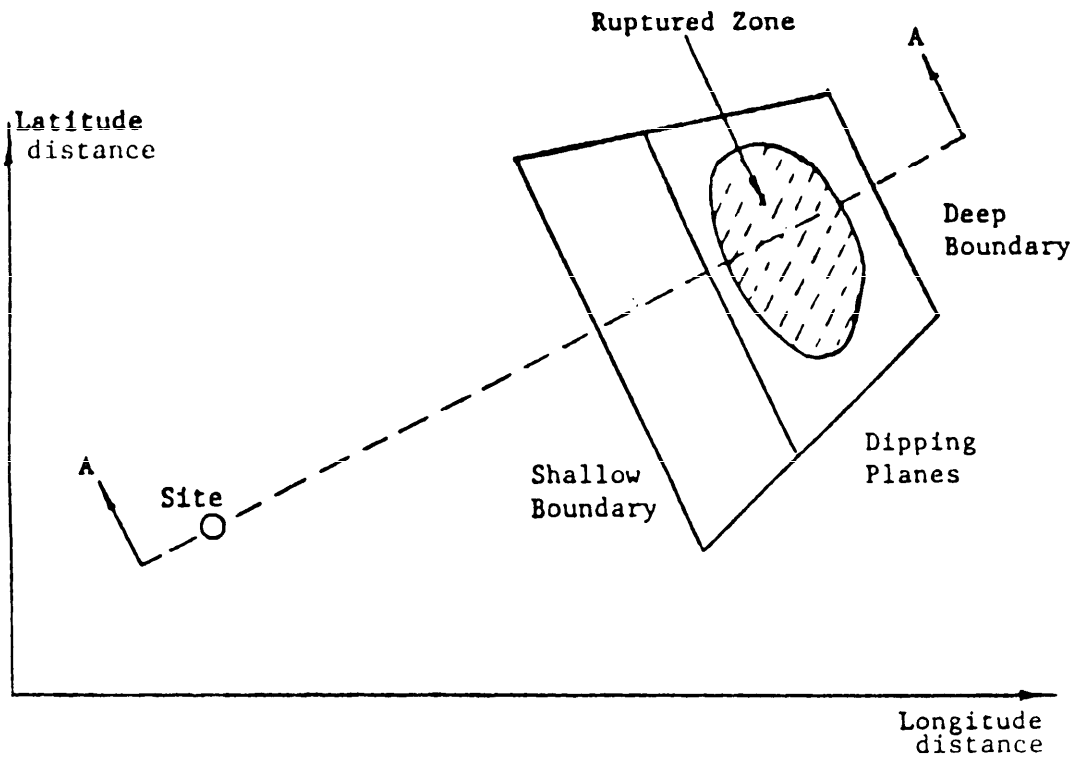
(b) Using subjective input as furnished by interviews from ten experts, Mortgat (Biblio 63 and 64), has developed homogeneous area sources as shown in figure 3-15. With respect to this method of using expert opinion, it is well to recognize that experts form their objective biases from the particular 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-



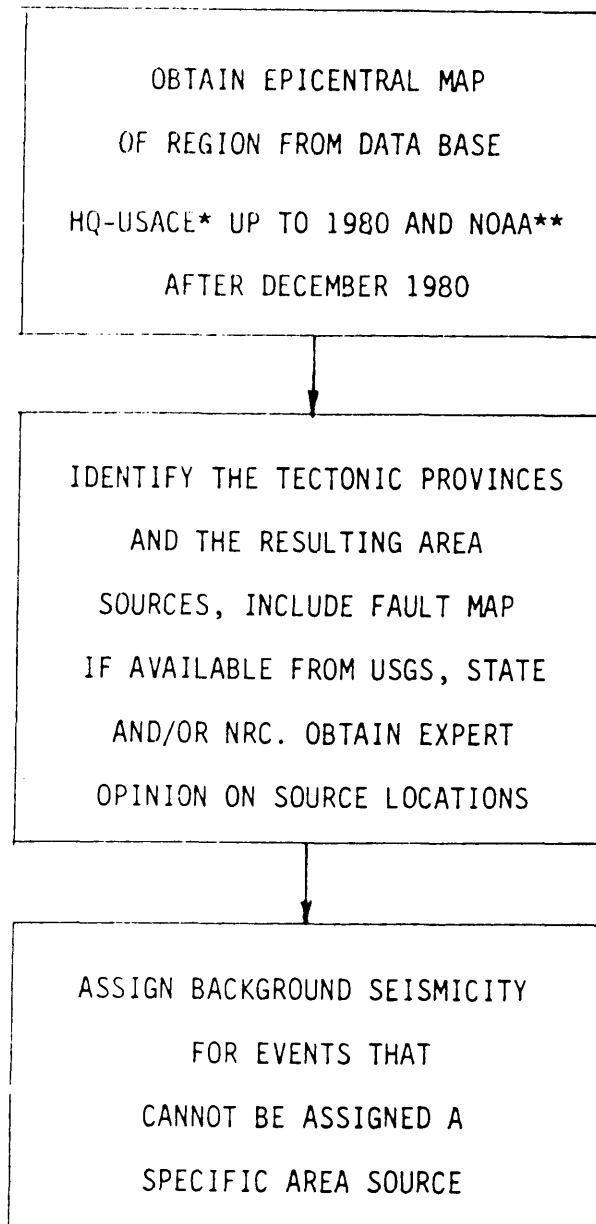
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Figure 3-10. Point, line and area sources.



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Figure 3-11. Dipping plane source.



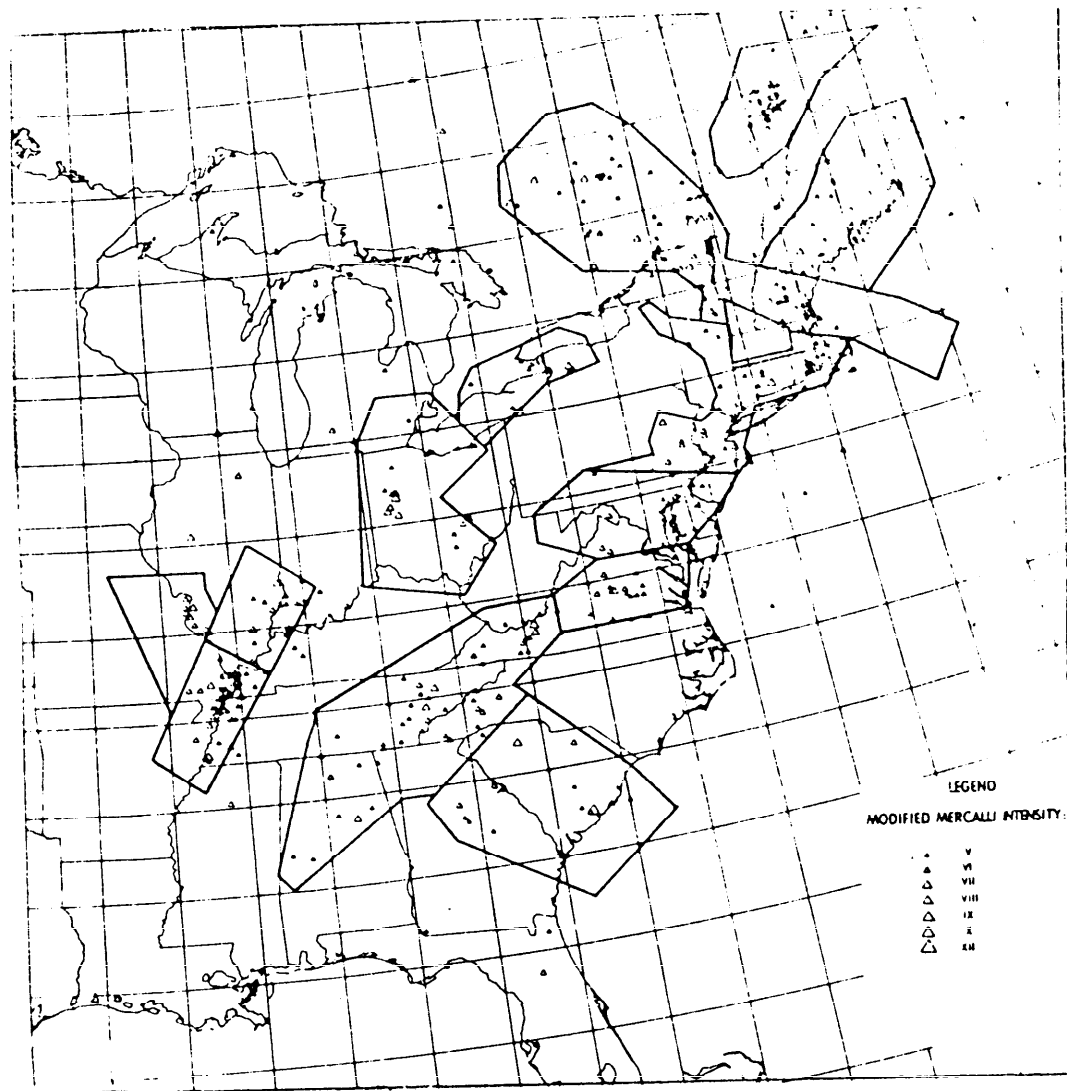
* DAEN-ECE-D Washington, D.C. 20314

** NOAA/NGSDC/TGB
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.

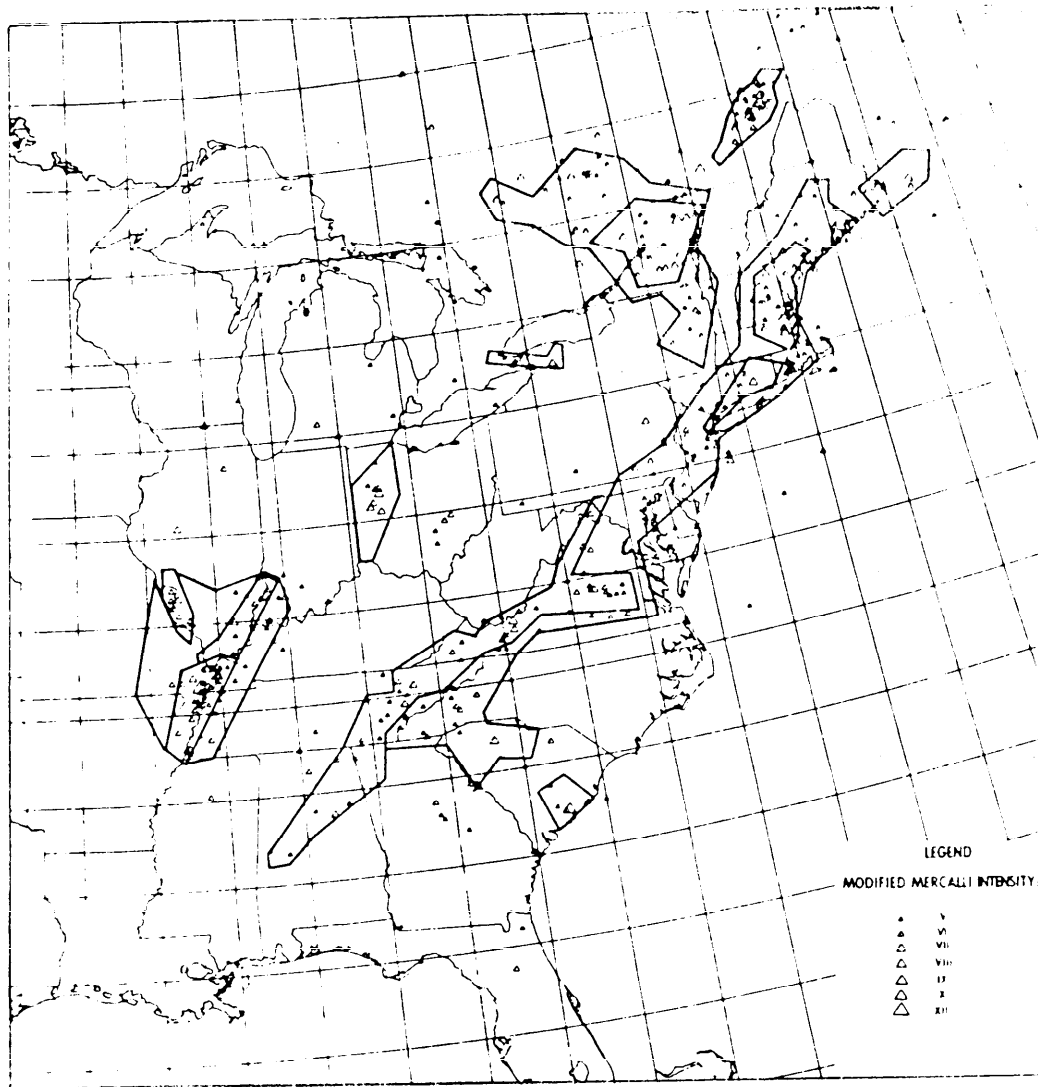
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Figure 3-12. Flow chart for step I source identification and modelling for the EUS.



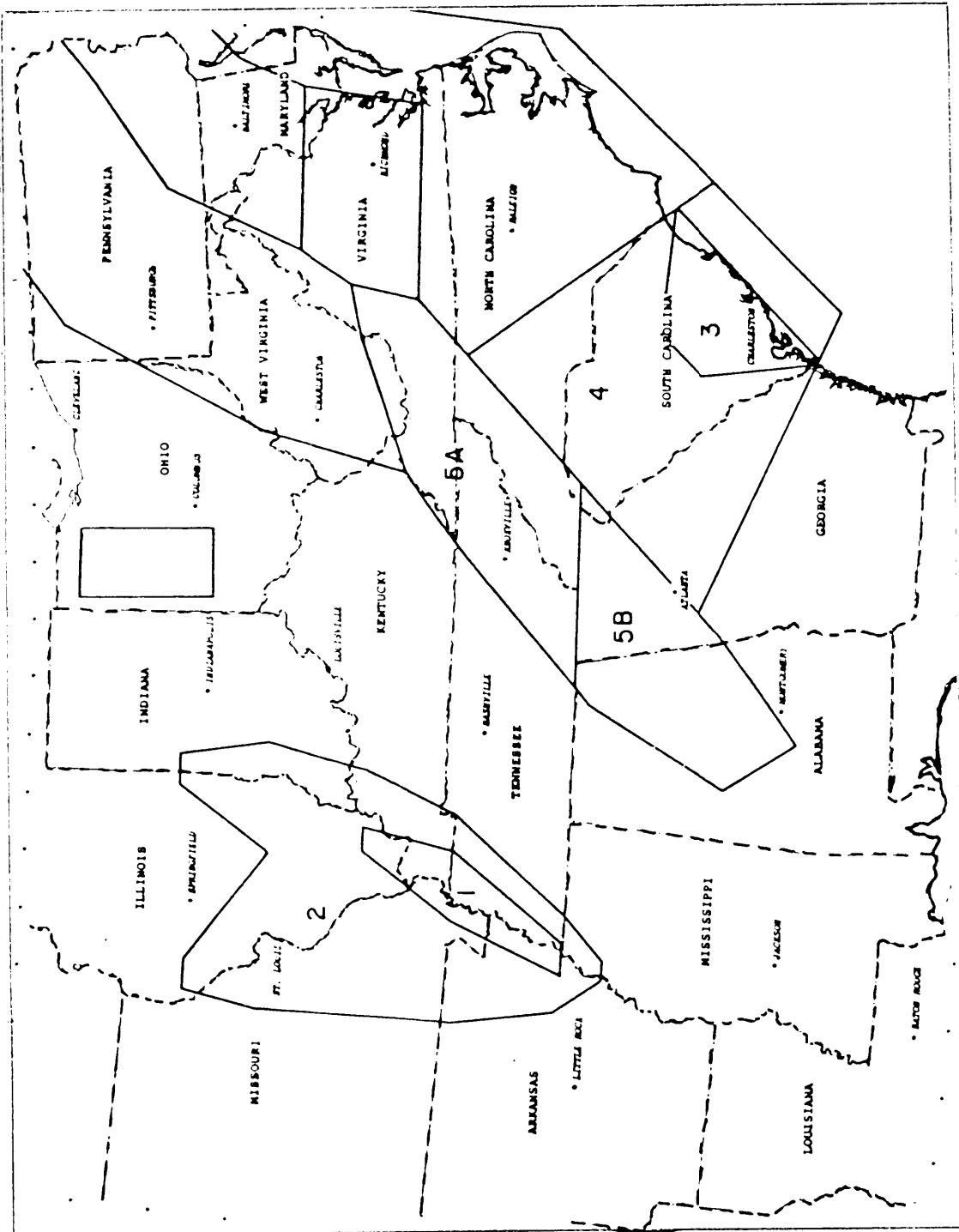
Reprinted from "Effects of Uncertainty in Seismicity on Estimates of Seismic Hazard for the East of the United States," McGuire, R. K., Bulletin of the Seismological Society of America, Vol. 67, No. 3, 1977, with permission from the Seismological Society of America.

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



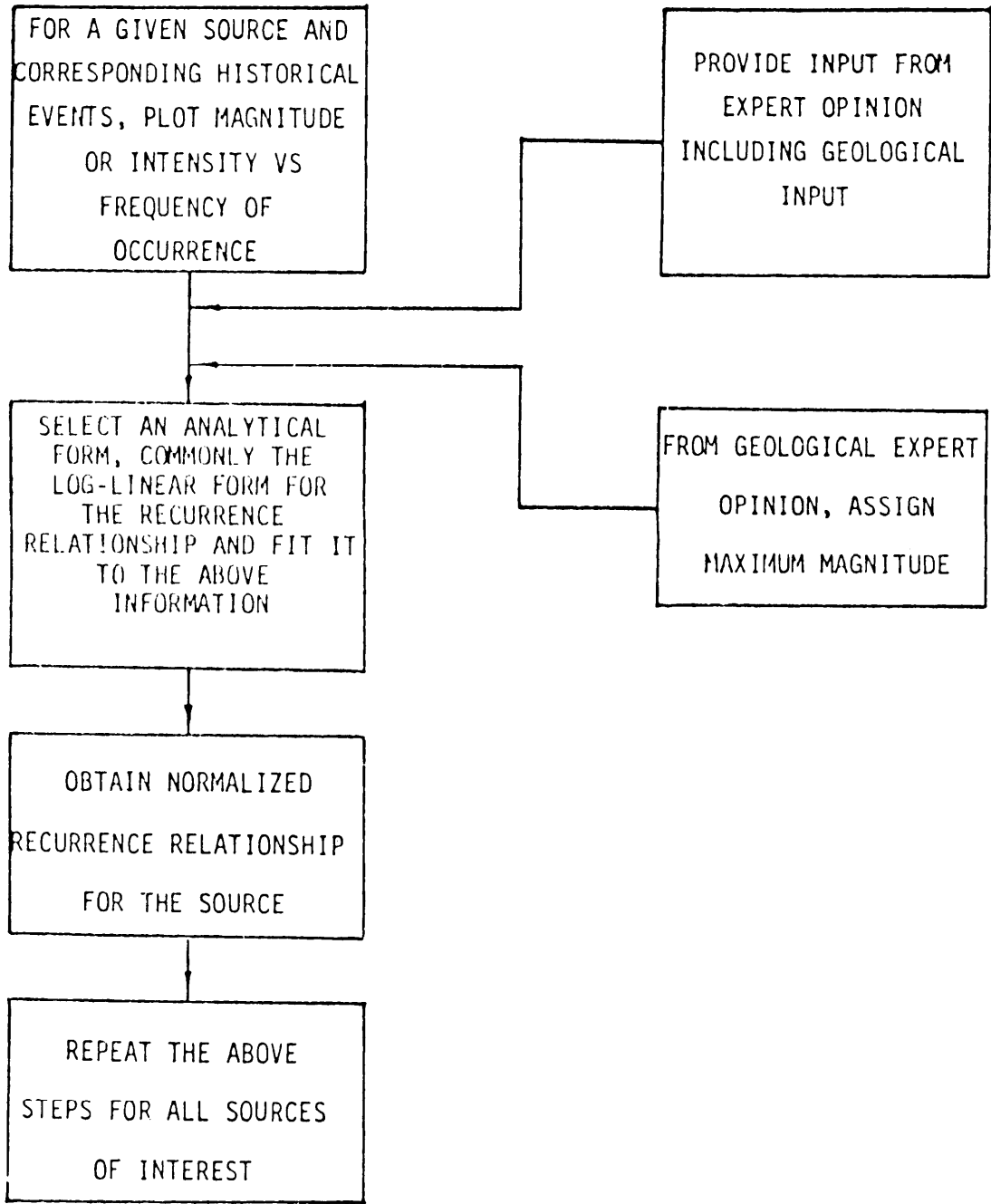
Reprinted from "Effects of Uncertainty in Seismicity on Estimates of Seismic Hazard for the East of the United States," McGuire, R. K., Bulletin of the Seismological Society of America, Vol. 67, No. 3, 1977, with permission from the Seismological Society of America.

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).

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

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.

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

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

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

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

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

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

$$\begin{aligned} \ln(N'(m)) &= \alpha + \beta m - \ln(LT) \\ &= \alpha' - \ln(LT) + \beta m \end{aligned}$$

or

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

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 similar format:

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

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

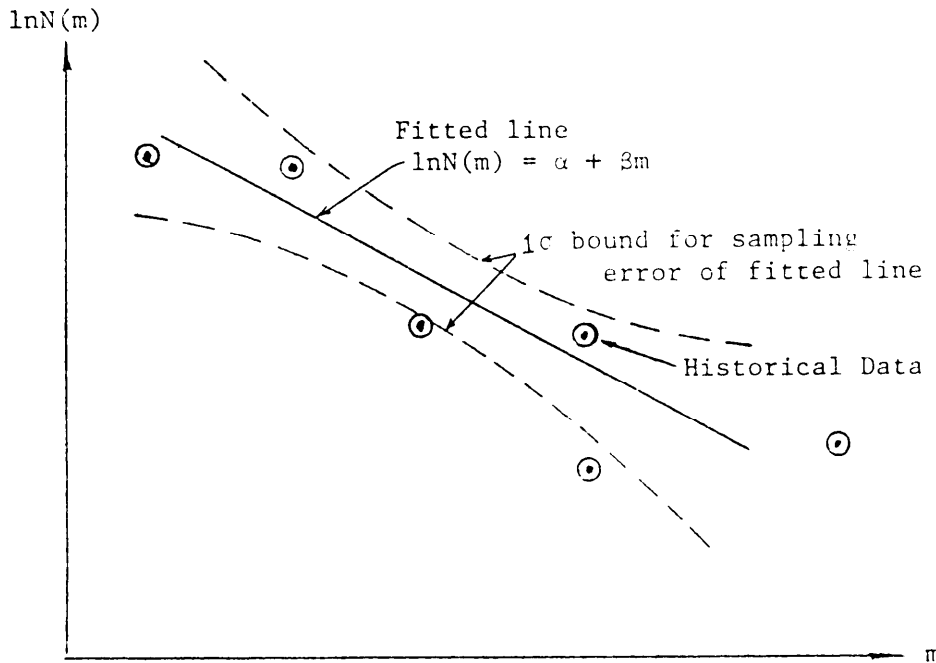
or

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

with $\alpha' = \alpha - \ln(AT)$
(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 3-4d. 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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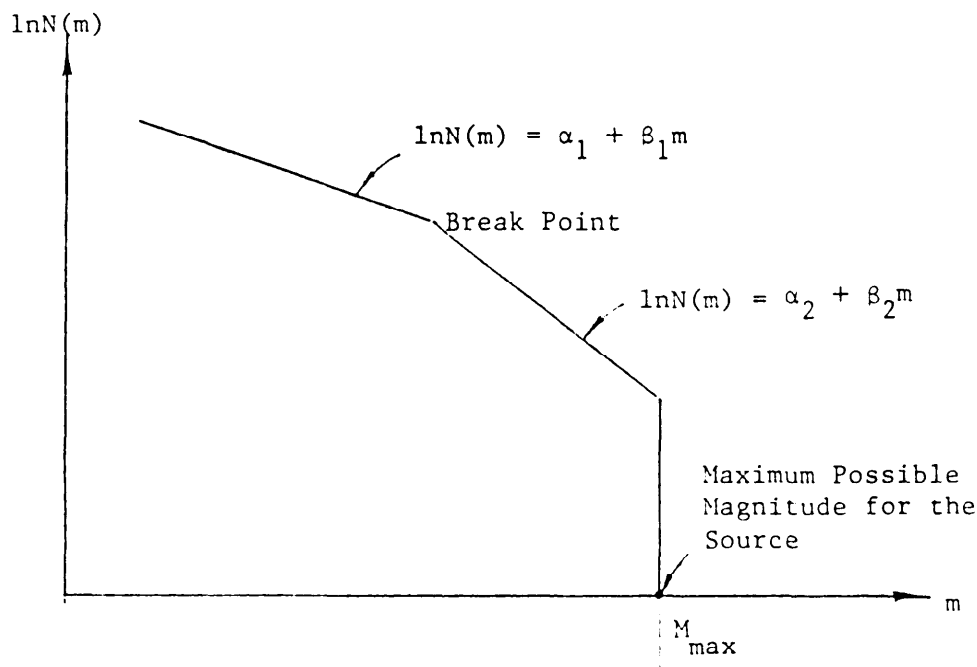
Figure 3-17. Linear Recurrence Relationship.

(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

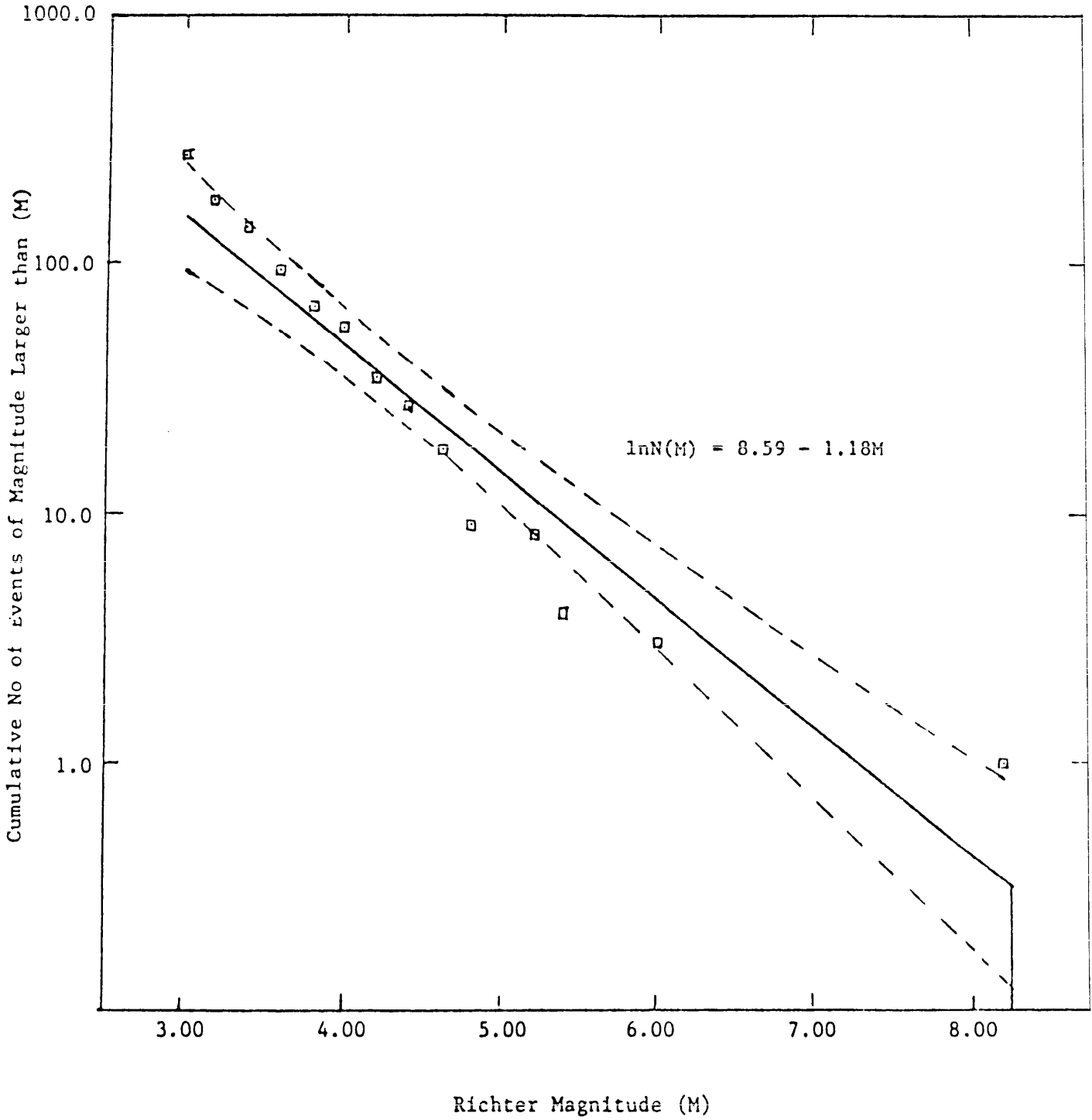
the recurrence constant for most of the WUS sources lies between about 1.1 to approximately 2.5. Figure 3-19 shows the recurrence relationship for the northern section of the San Andreas Fault in California. It should be mentioned here that one large fault such as the San Andreas may be broken down into 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 ,

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

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:

$$\ln[N(I_o)] = \alpha_1 + \beta_1 I_o \quad (\text{eq 3-8})$$

Where α_1 and β_1 are regression constants. Ye-gian (Biblio 71) and TERA (Biblio 63,64) have given values of β_1 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_1 value applies throughout very large regions and that local variations apply only to the level of seismicity (parameter α_1). The range of values for the parameter β_1 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-

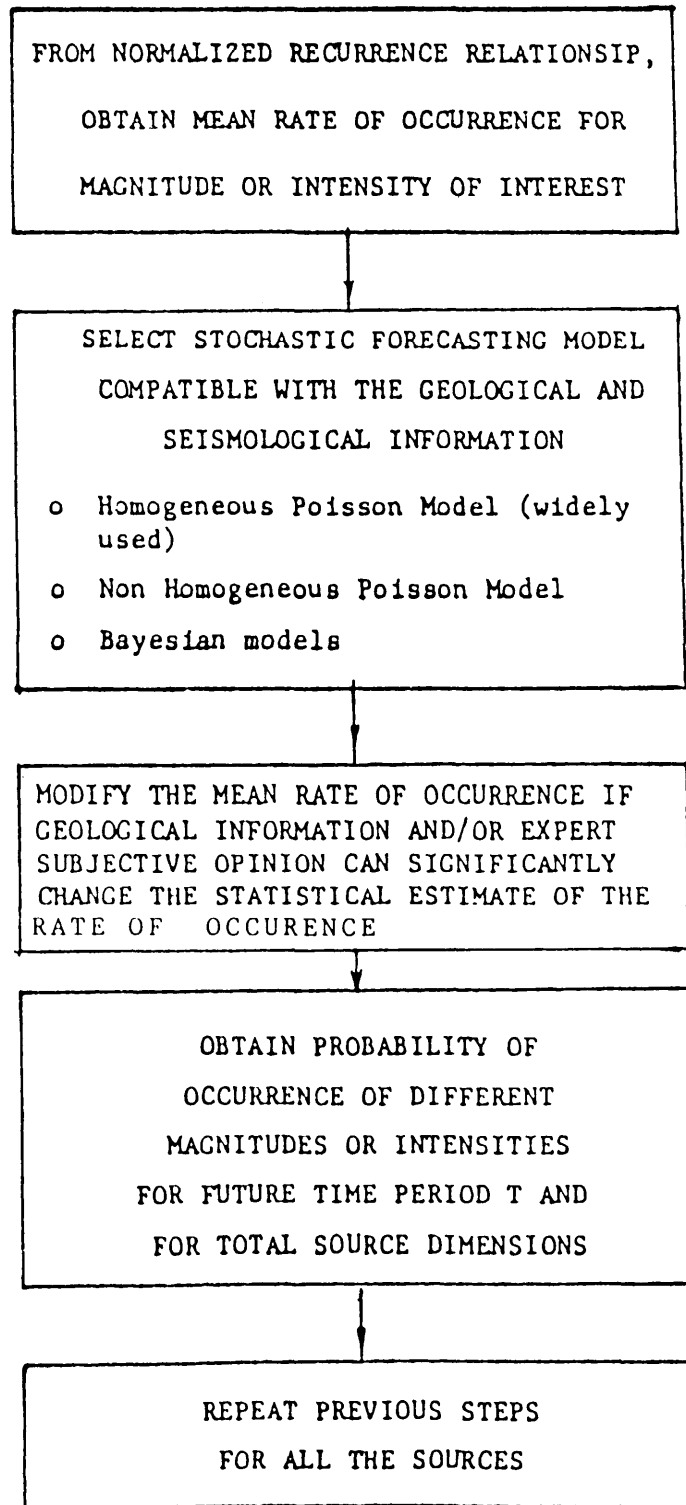
lio 40)) seismic moment has been related to the fault rupture area, along with the fault shear modulus and average slip. Relating the maximum seismic moment $M_{o,max}$ to moment magnitude M_m gives the value of the largest moment magnitude. It is useful to note that M_m is equal to M_L for M_L values 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, 3-4d.

(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 corre-

sponding 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 3-4c(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 3-4b) and the seismicity of the identified sources has been determined (para 3-4c). 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), Kirimidjian 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-homogeneous 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

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

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,

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

= 0, 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

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

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

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

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

Thus,

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

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

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

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

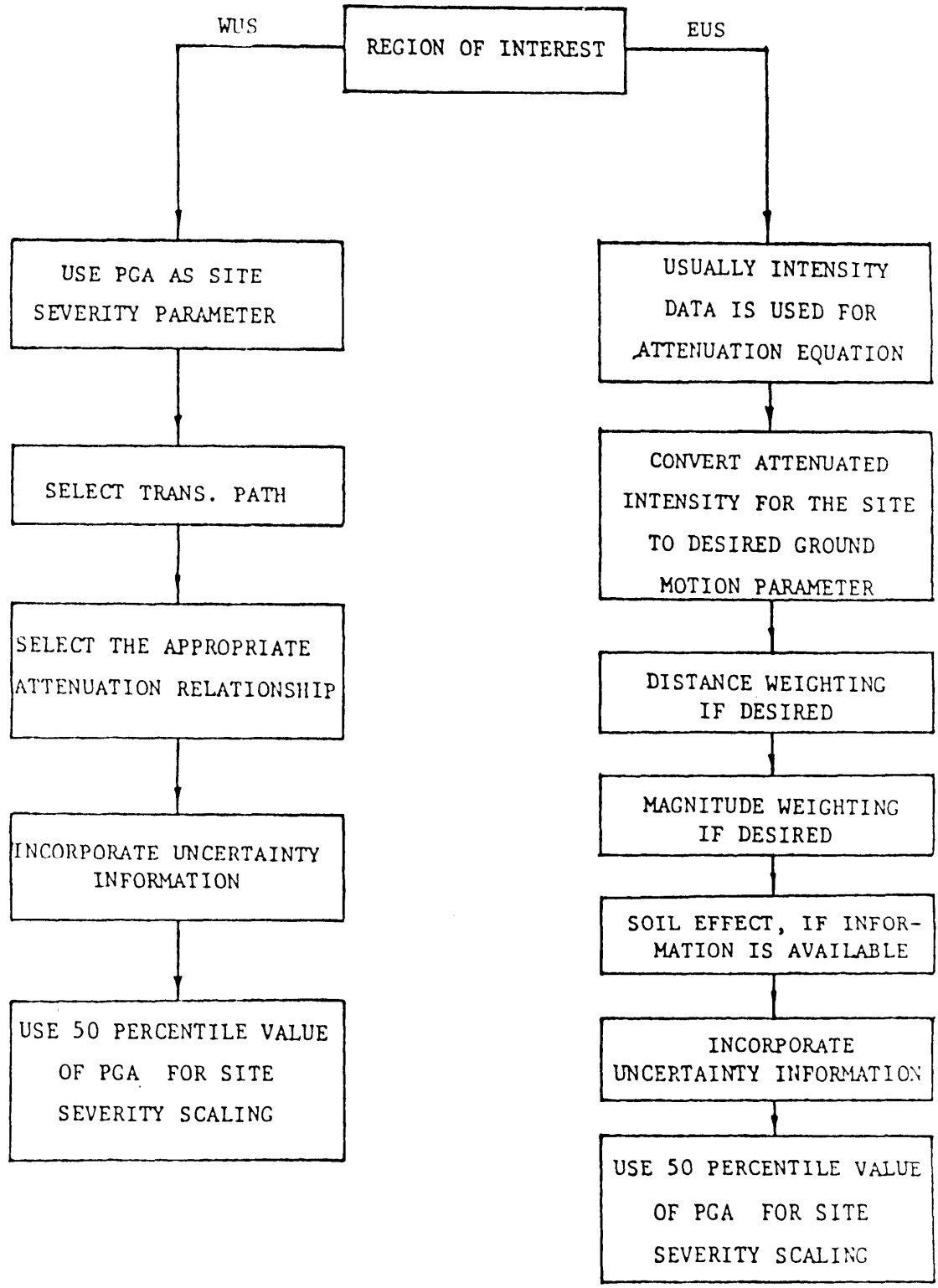
In estimating the type and severity of ground motion that would exist at a site due to some future seismic event, the analyst should incorporate the above parameters in his model. The current state-of-the-art methods for estimating the ground motion can be classified into two groups.

- 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 com-

ponent 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 non-uniformity 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-

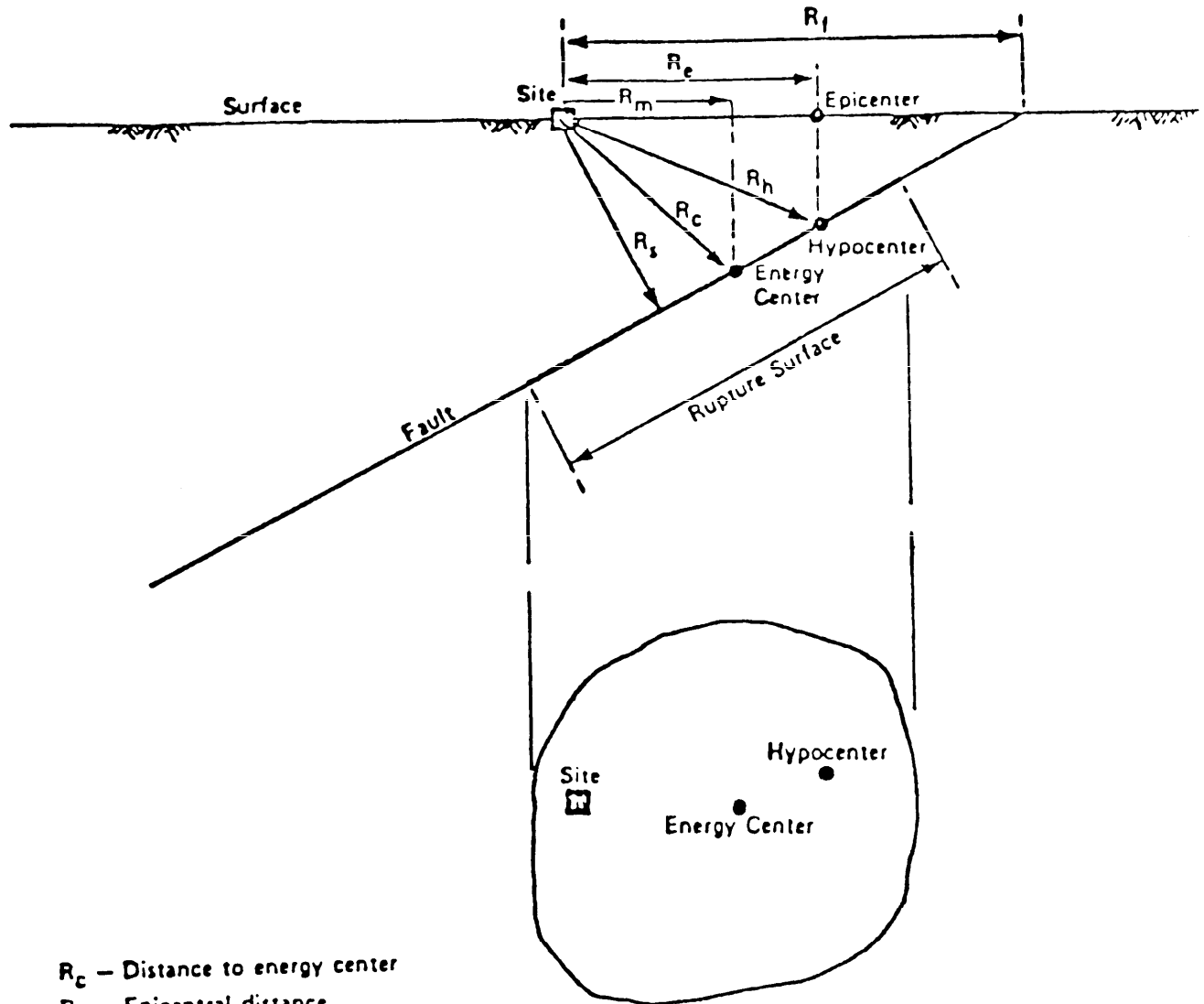


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Figure 3-21. Step IV, attenuation of ground motion from source to site.

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

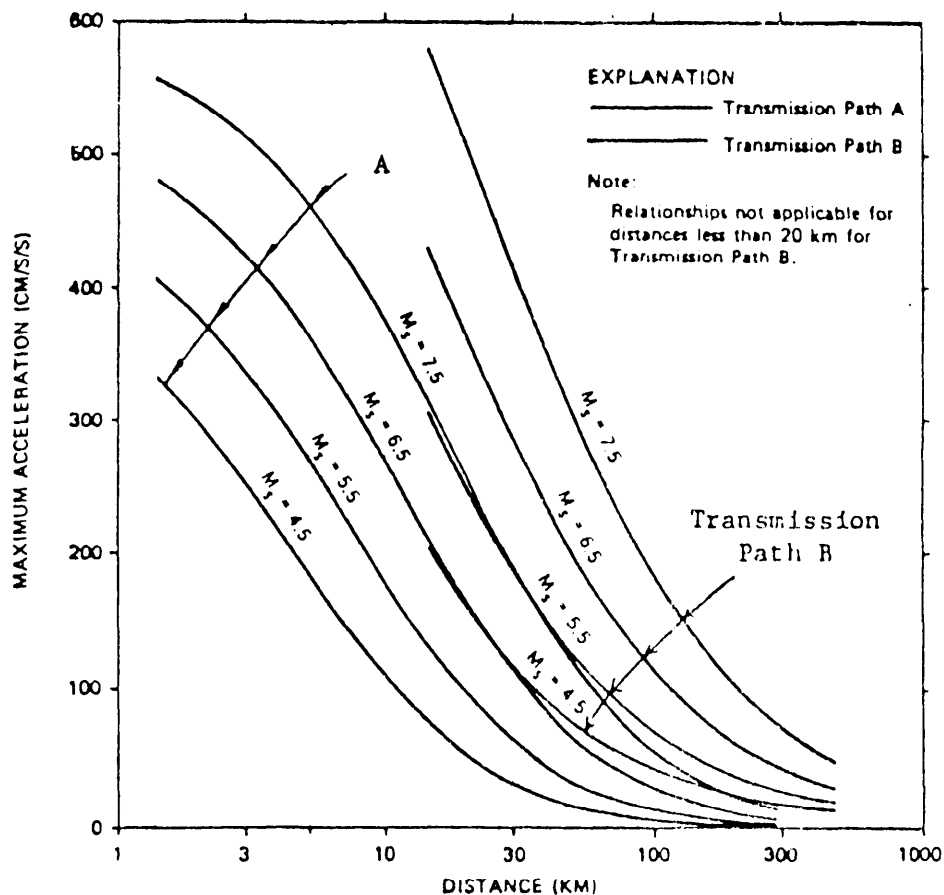
Reprinted from "Offshore Alaska Seismic Exposure Study (OASES)," 1978, with permission from Woodward-Clyde Consultants.

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)

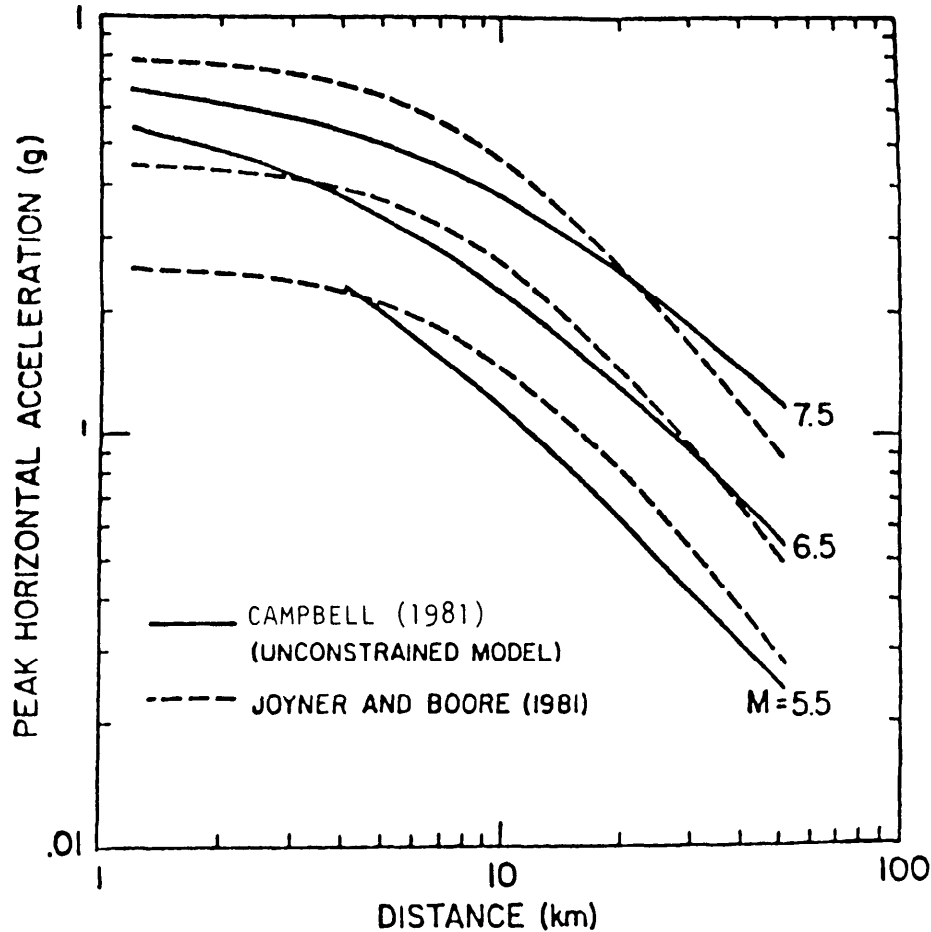
Figure 3-24 shows the first two of these relationships. The third relationship is given in figure 3-23.

(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:

$$PGA = b_1 \exp(b_2 M)(R + C)^{b_3} \quad (\text{eq 3-21})$$

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.

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

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

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

$$\text{Log GM} = g(I_s, R) \text{ and in some cases } G(I_s, M_L, R) \quad (\text{WUS D})$$

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

Reprinted from "Offshore Alaska Seismic Exposure Study (OASES)," 1975, with permission from Woodward-Clyde Consultants.

Magnitude weighting

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

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

No weighting

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

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

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:

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

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:

$$\ln GM = C_1 + C_2 I_s + C_3 \ln R \quad (\text{eq 3-24a})$$

or

$$\ln GM = C_1 + C_2 I_s + C_3 M \quad (\text{eq 3-24b})$$

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, $C_4 S$ ($S = 0$ for soil sites and $S = 1$ for rock sites) where C_4 is obtained from WUS data, ($\ln(GM) = C_4 S + C_1 + C_2 M_L + C_3 \ln R$). 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. Ossipee 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:

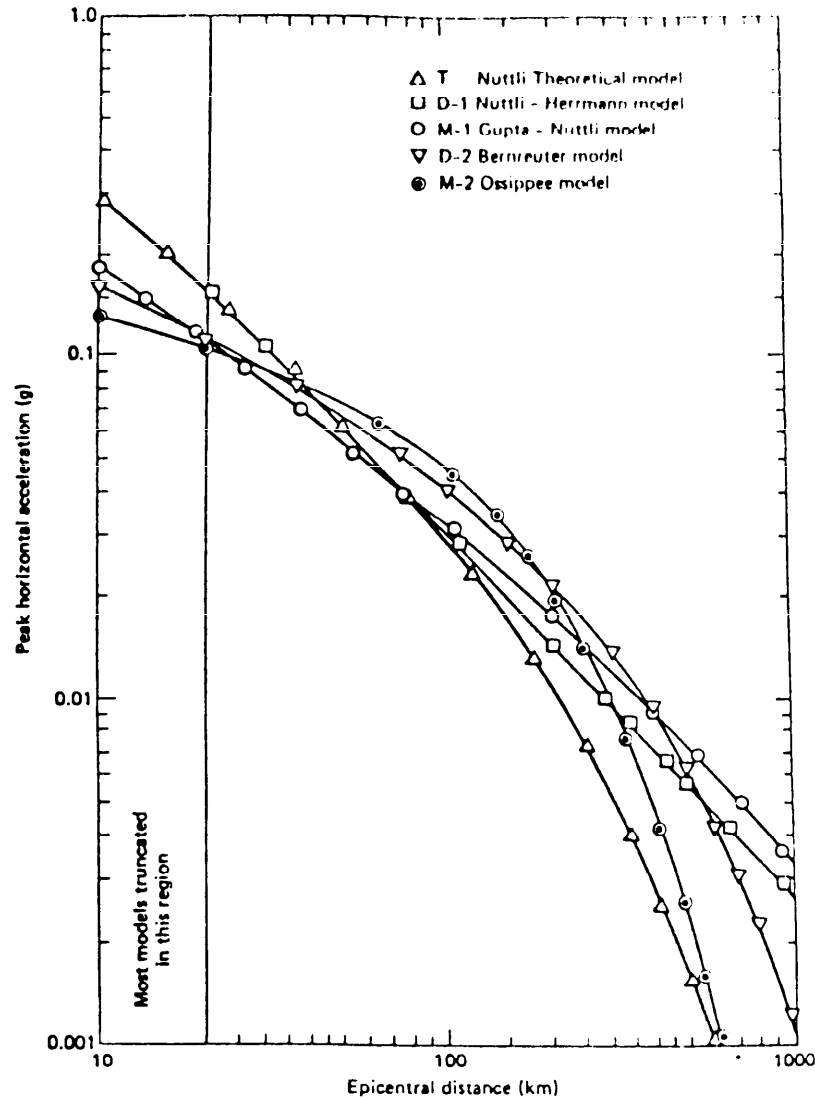
$$\begin{aligned} \log \text{PGA} &= 0.74 + 1.12 m_b - 0.733 \ln R - \\ &0.0007R \\ &(\text{for } R > 20 \text{ kilometers.}) \\ &= -1.47 + 1.12 m_b \\ &(\text{for } R \leq 20 \text{ kilometers.}) \end{aligned}$$

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.

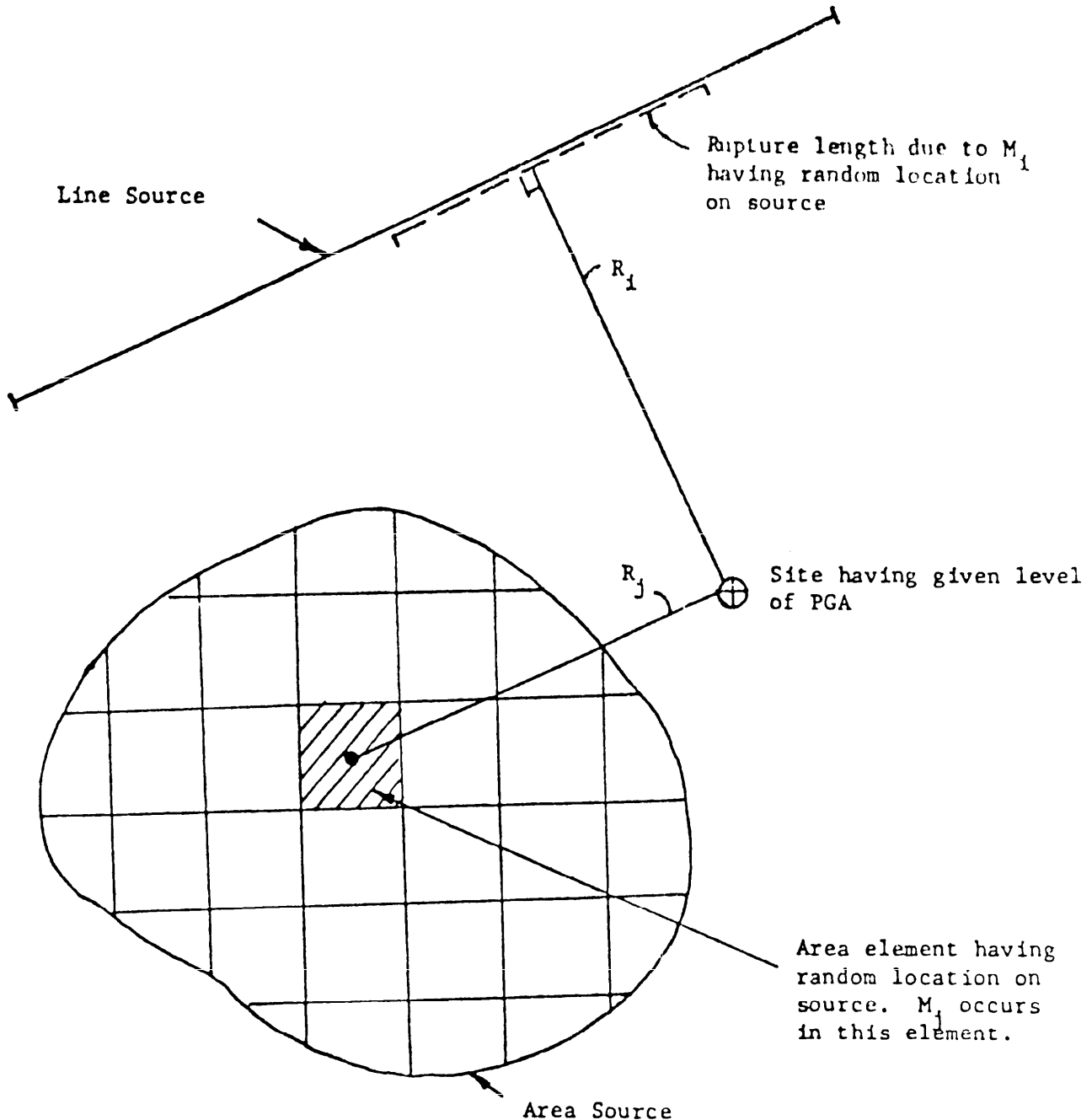
f. Site severity for scaling the response spectrum shape. For the purpose of scaling the appropriate site response spectrum shape (DAF) as described in the next paragraph 3-6, it is recommended that the median or 50 percentile value of PGA be used in the attenuation equation. The mean value shall be 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 3-7c.

(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) judgmentally 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.

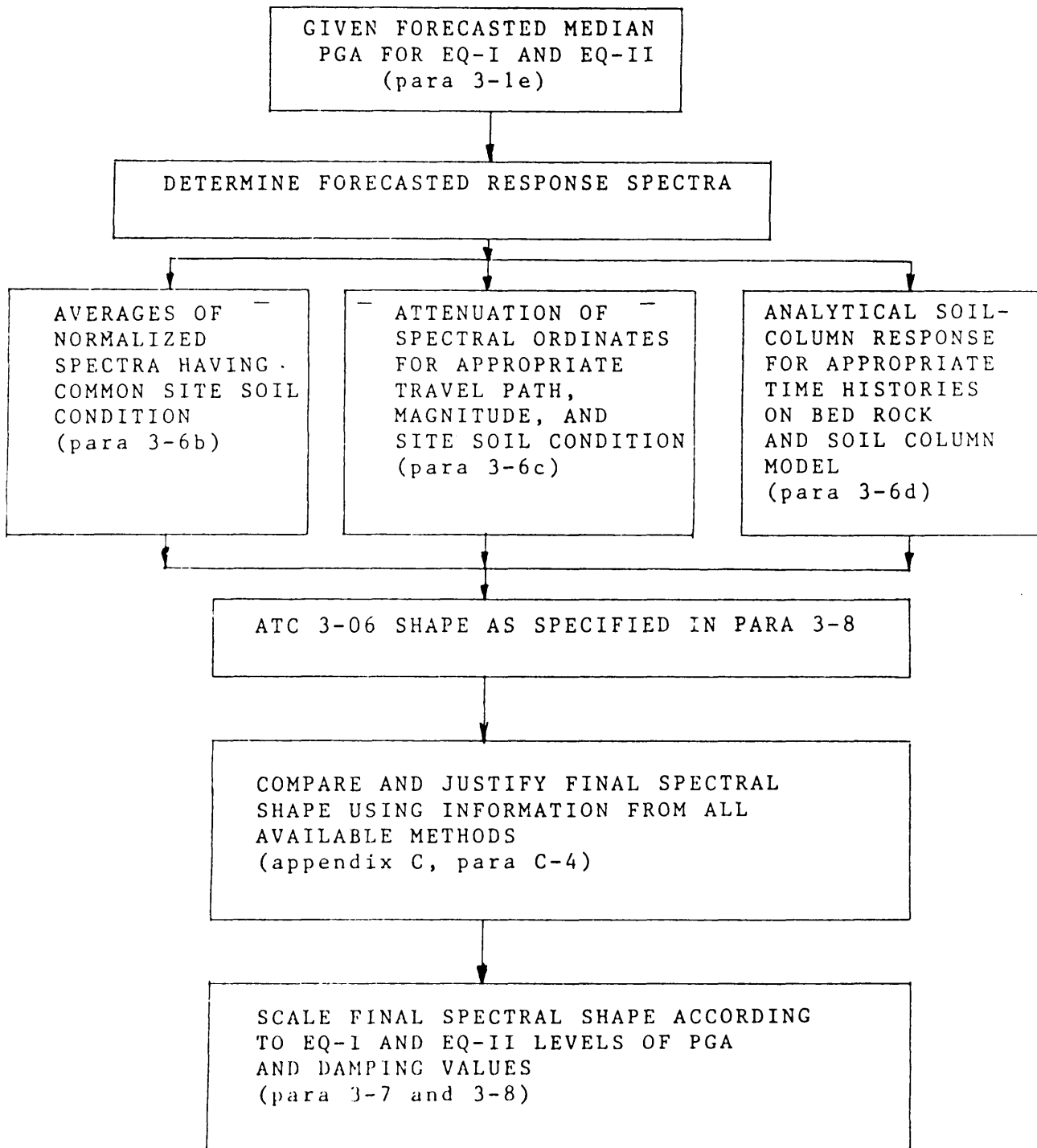
a. Spectral shape factors. It is generally recognized that the frequency content and corresponding response spectrum shape is governed by the following source and site factors.

- 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.

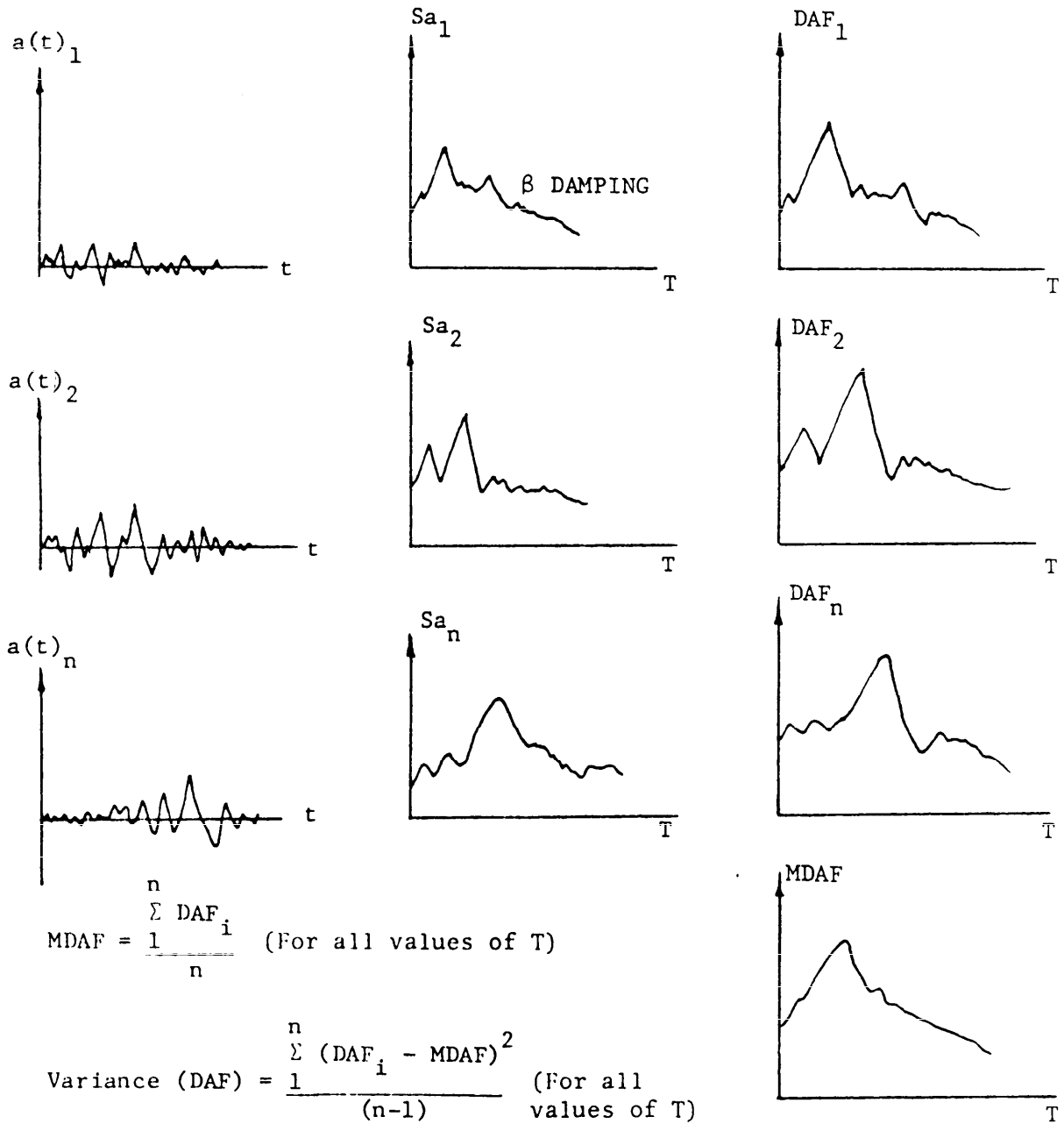
b. Statistical averages of normalized response spectra. In this first empirical method, the shape of the spectrum is determined by a statistical analysis (evaluation of averages and standard deviations) of past earthquake strong motion accelerograms; as classified according to site conditions, distance 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.



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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-3d). 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 3-6b, 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)

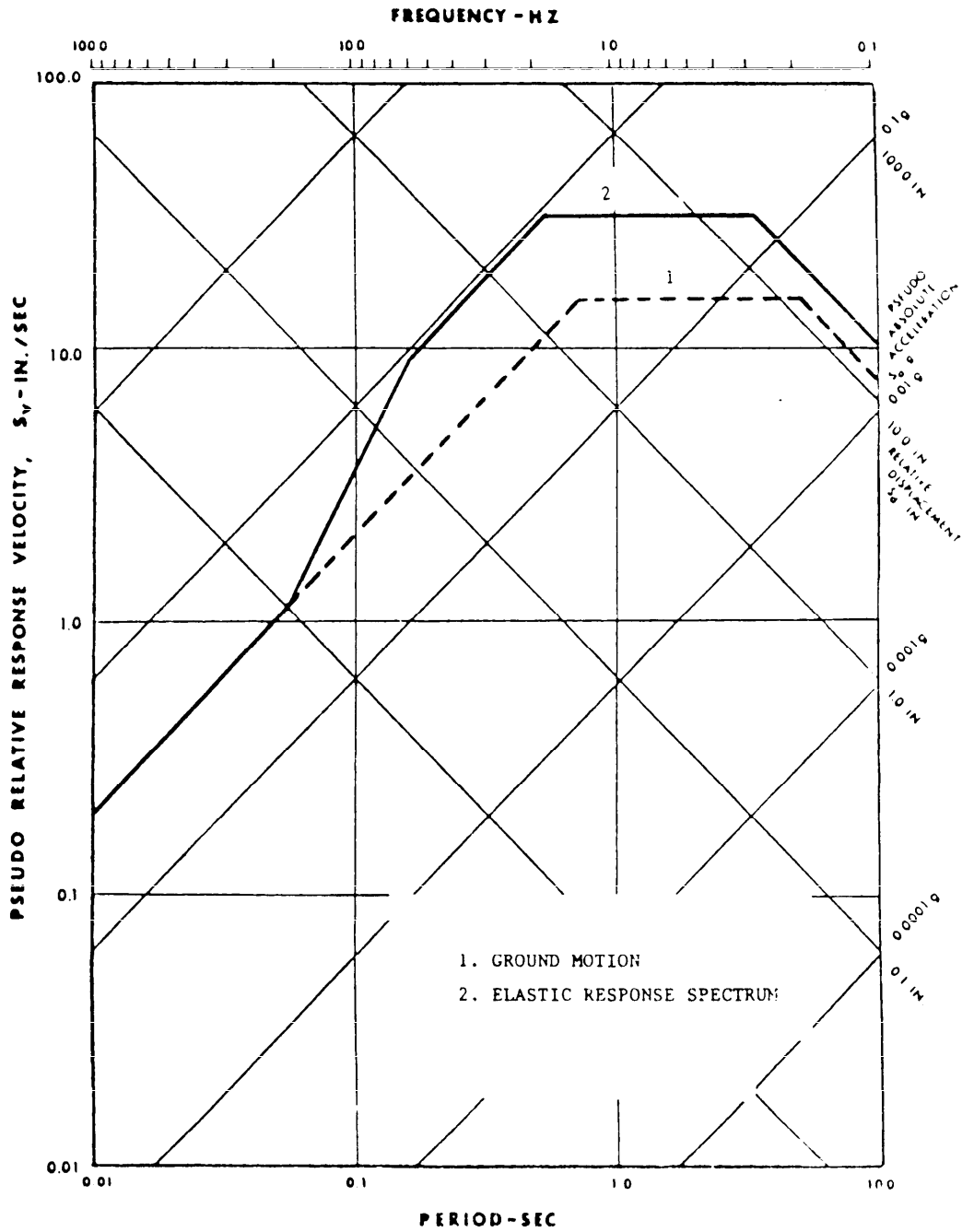
Draw this ground motion spectrum on the tripartite paper. (fig 3-29).

(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, % Critical	One Sigma (84.1%)			Median (50%)		
	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.

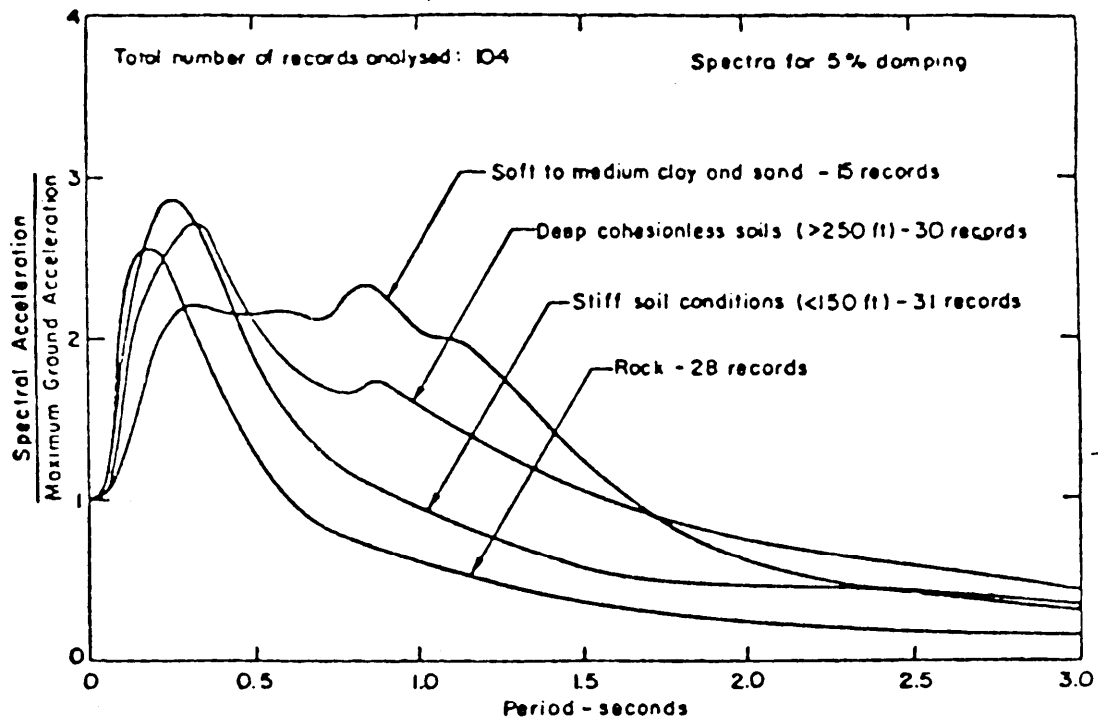
(2) *Seed et al.* This method provides mean DAF, and mean plus one standard deviation shapes for different categories of site conditions, see figures 3-30 and 3-31. These DAF shapes may be scaled to the forecasted PGA value having a given risk value at the site.

(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 3-6a 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.

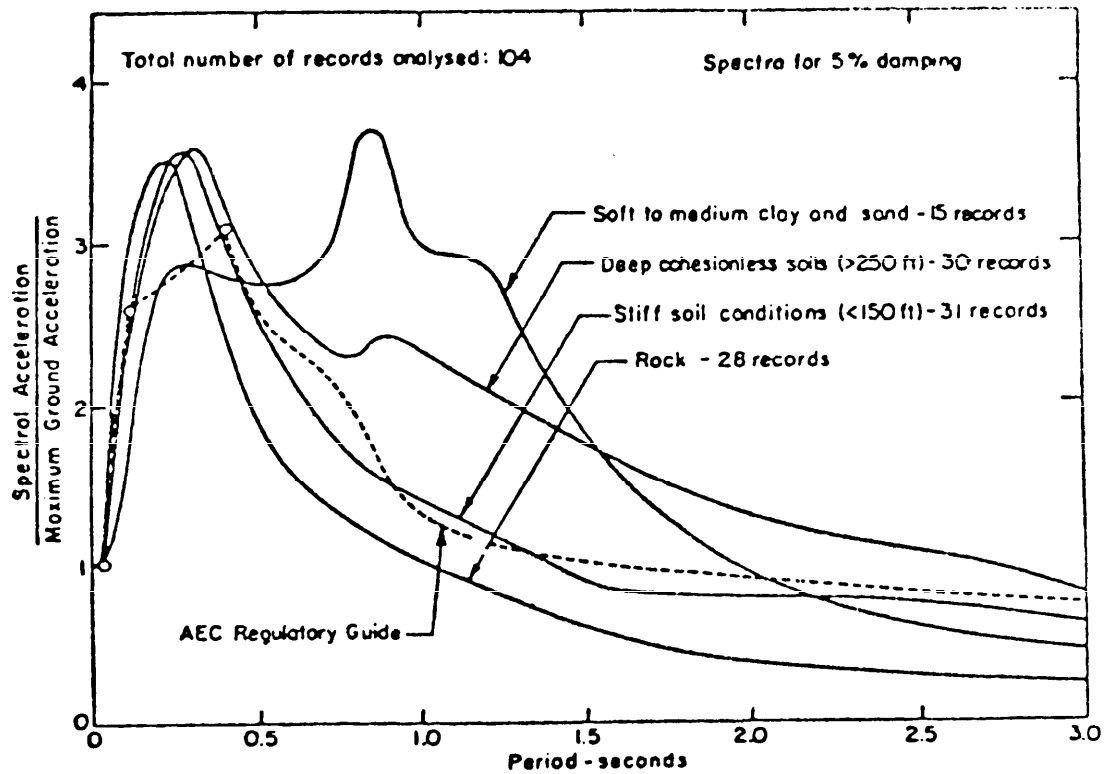
Figure 3-30. Average acceleration spectra for different site conditions.

(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

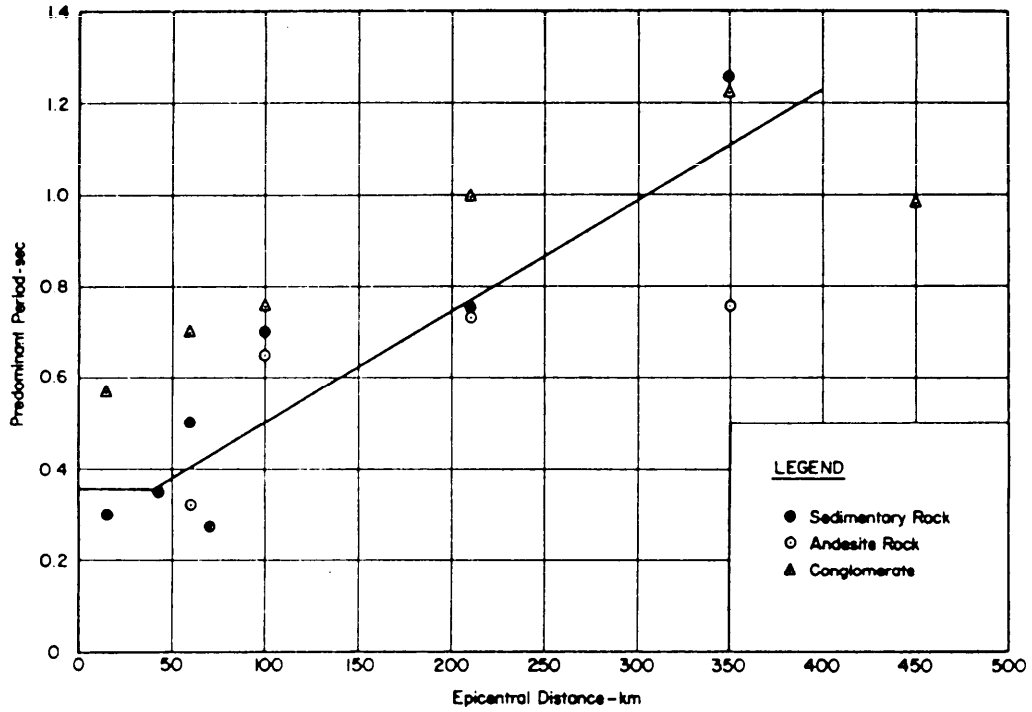
source to the site increases. In other words, the predominant period of motion increases with distance and size of the seismic event. The engineering implication of this observation is obvious. Taller structures are affected more by large distant earthquakes than are the shorter (or stiffer) structures 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



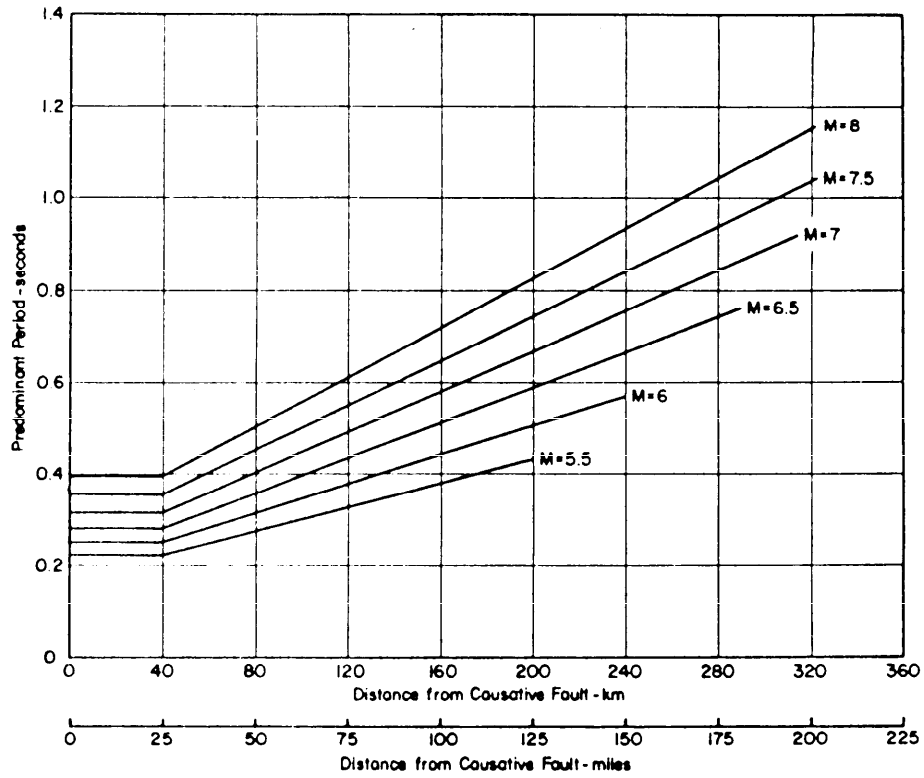
Reprinted from "Site-Dependent Spectra for Earthquake Resistant Design," Seed, H. B. et al, Report No. EERC 74-12, University of California at Berkeley, 1974.

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.

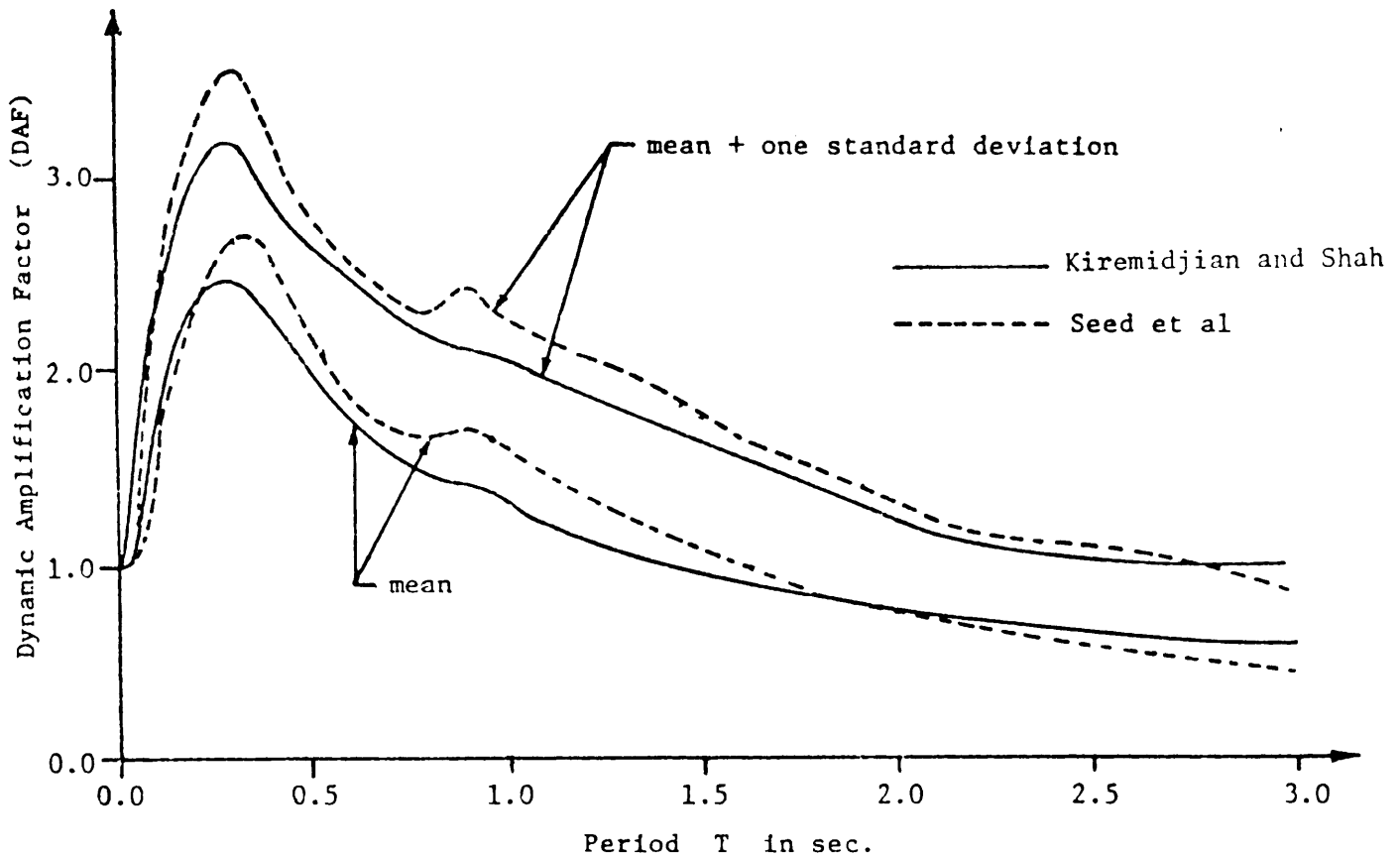
classification, the following procedure is recommended (V_s is the shear wave velocity):

- 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

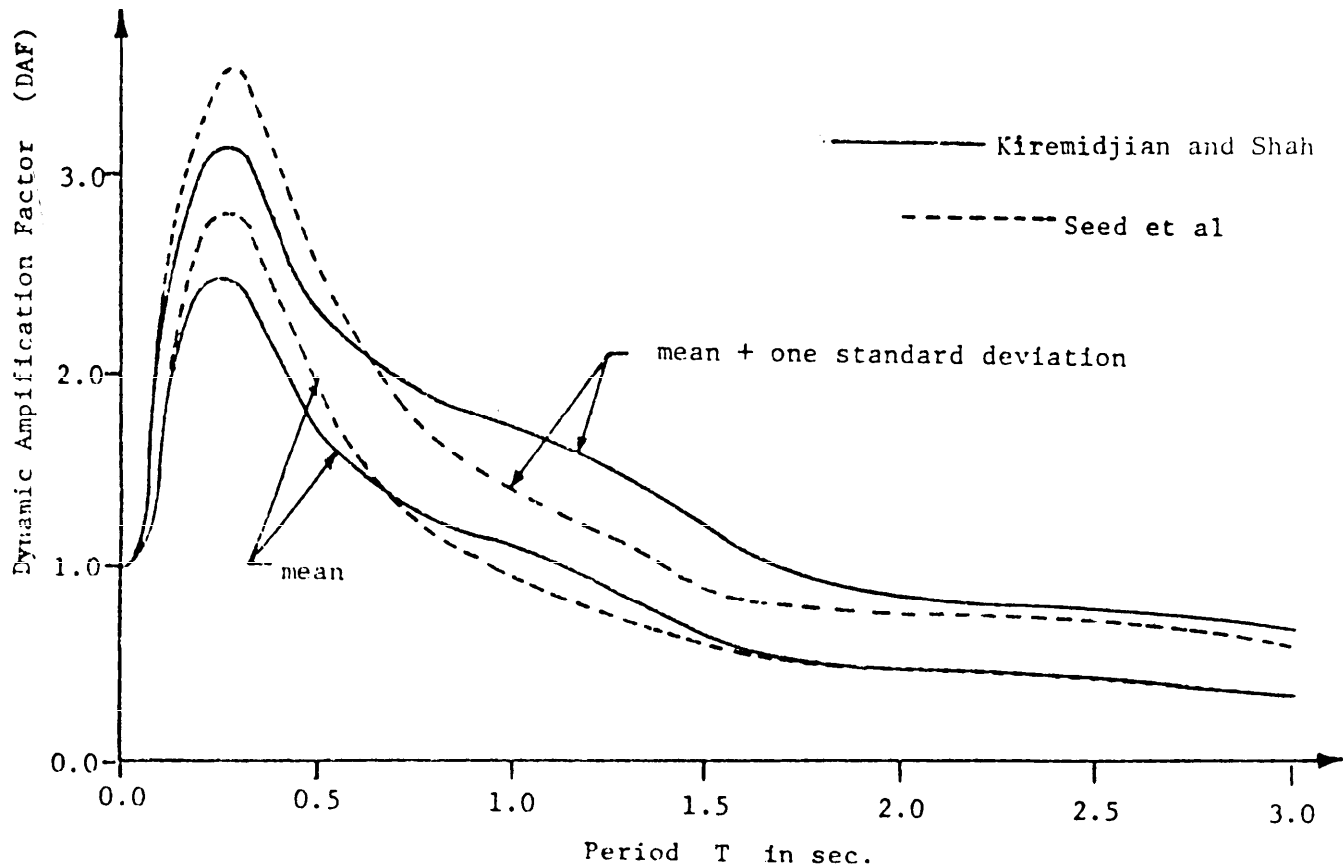
soils of class 2. Under very special conditions, (such as in Mexico City, where the city is on an old lake bed), the spectral peak could occur at a period as long as 1.5 to 2.5 seconds.

(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



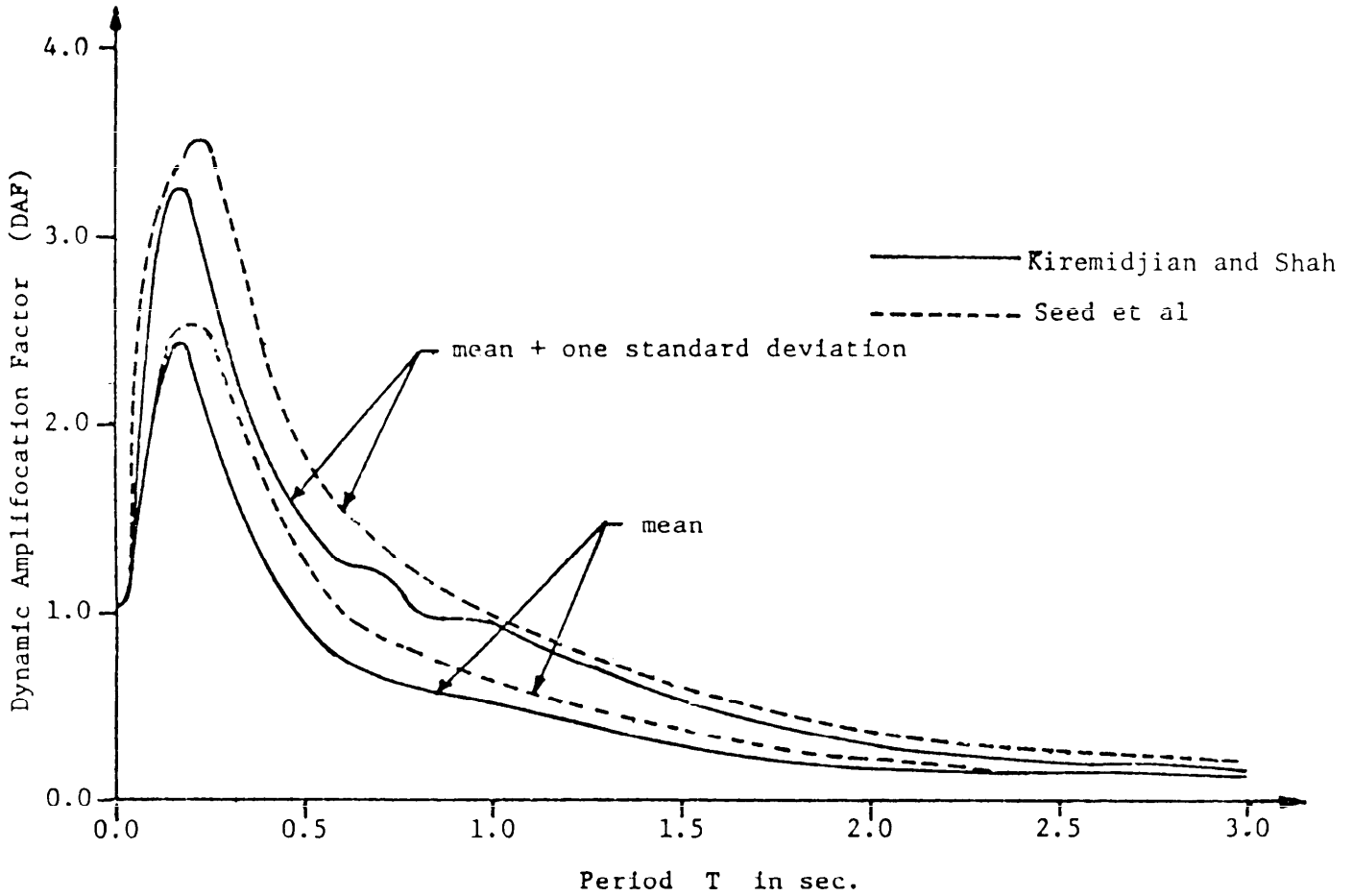
Reprinted from "Probabilistic Site-Dependent Response Spectra," Kiremidjian, A. S. and Shah, H. C., Journal of the Structural Division, Proceedings of the ASCE, Vol. 106, No. ST1, January 1980, with permission from the American Society of Civil Engineers.

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%.



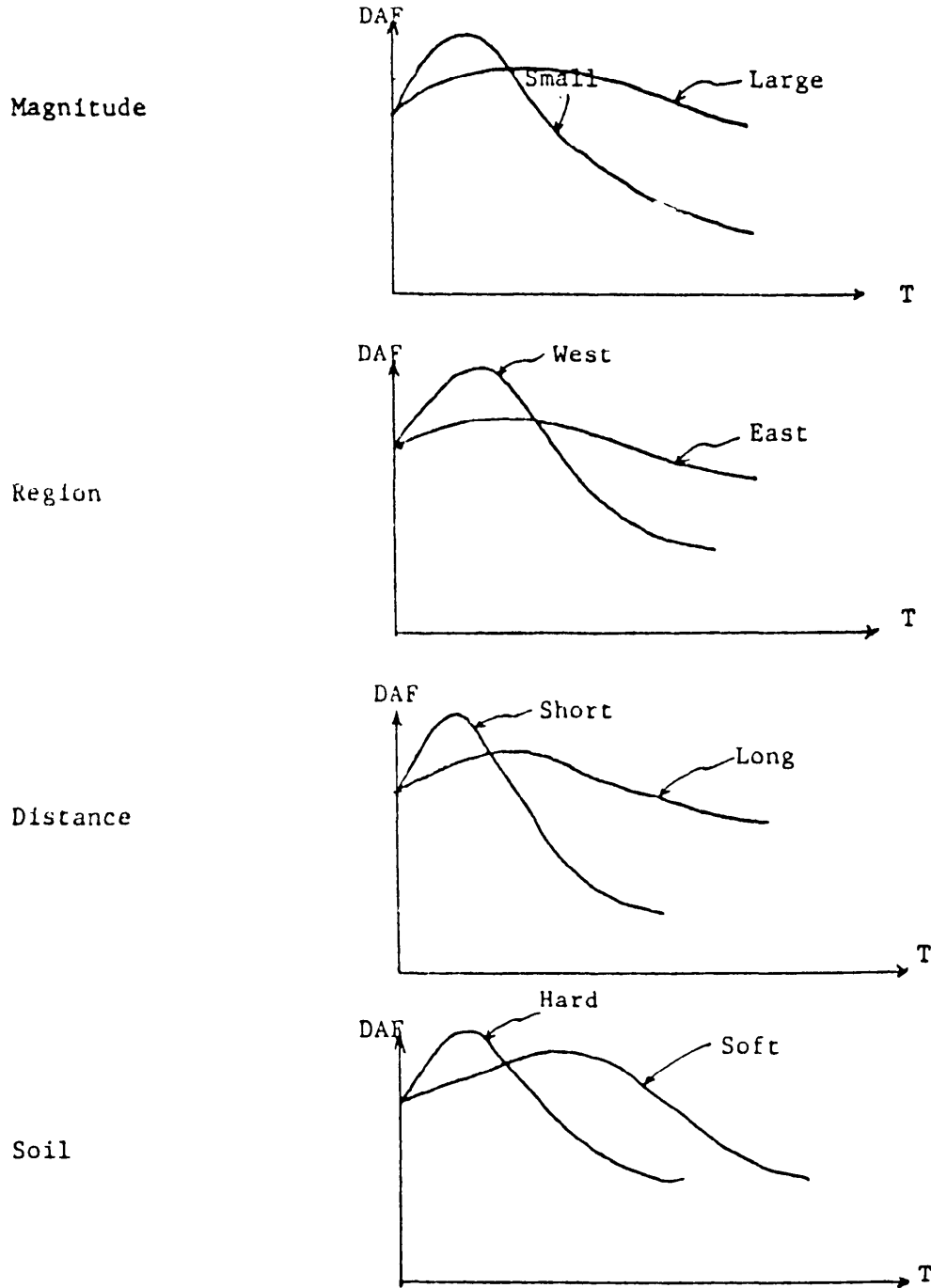
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Figure 3-36. Comparison of DAF from present study to DAF from Seed et al, soil class = 2, damping = 5%.

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-3f, appendix C.

h. Effective response spectra. In paragraph 3-8e, 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 3-6g, 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 3-8e.

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.

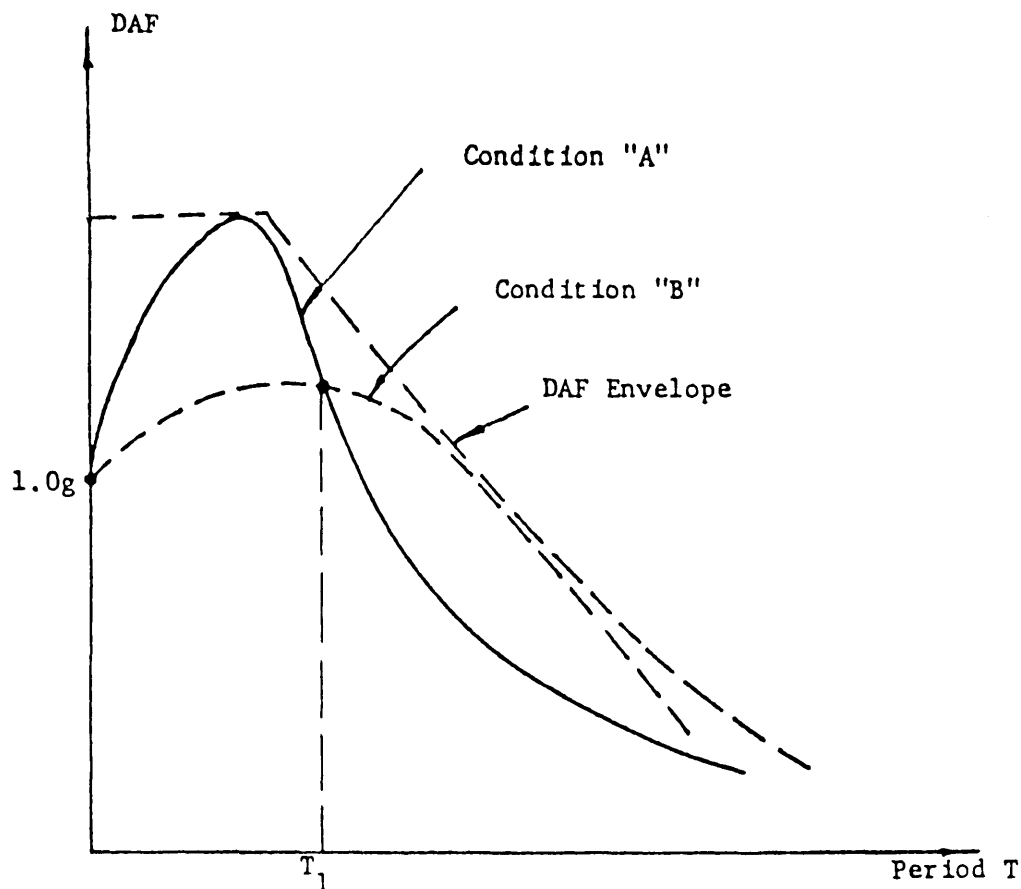
(1) *Site severity (PGA).* Given the acceptable hazard in terms of the return period (T_R)

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.

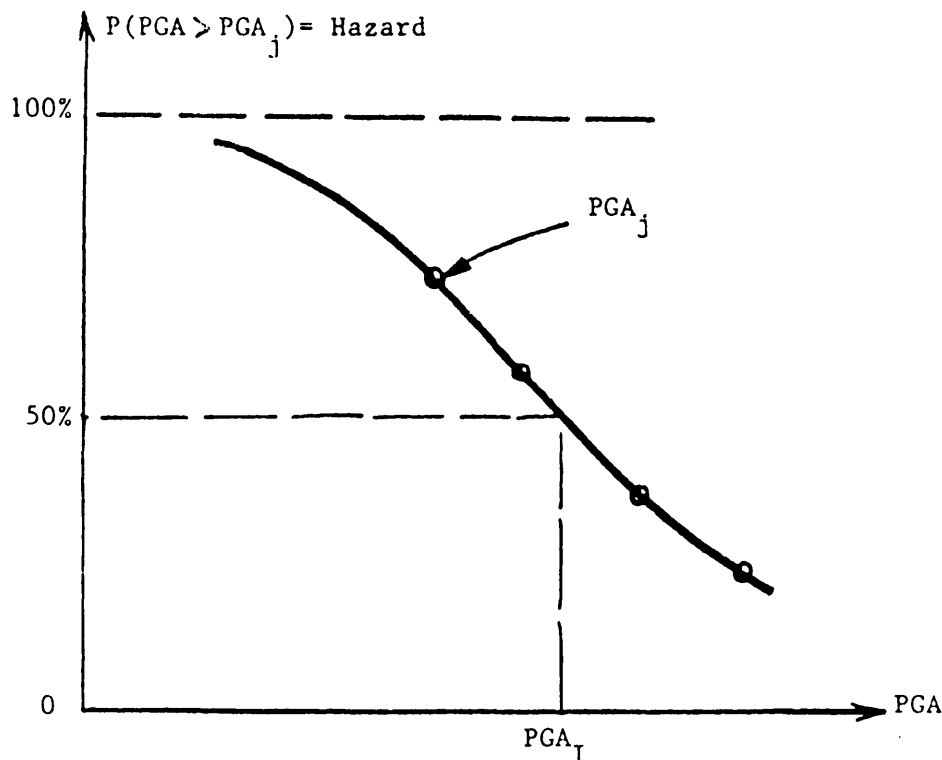
(2) The effects or consequences of uncertainties and variabilities in specified ground motion values for a site are best evaluated after the consideration of the total structural design process in chapter 4. When the forces and deformations in the structural model have been evaluated for the specified 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

$$S_a = \text{PGA} \times \text{DAF},$$

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 $P[\text{PGA} > \text{PGA}_j]$ 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.

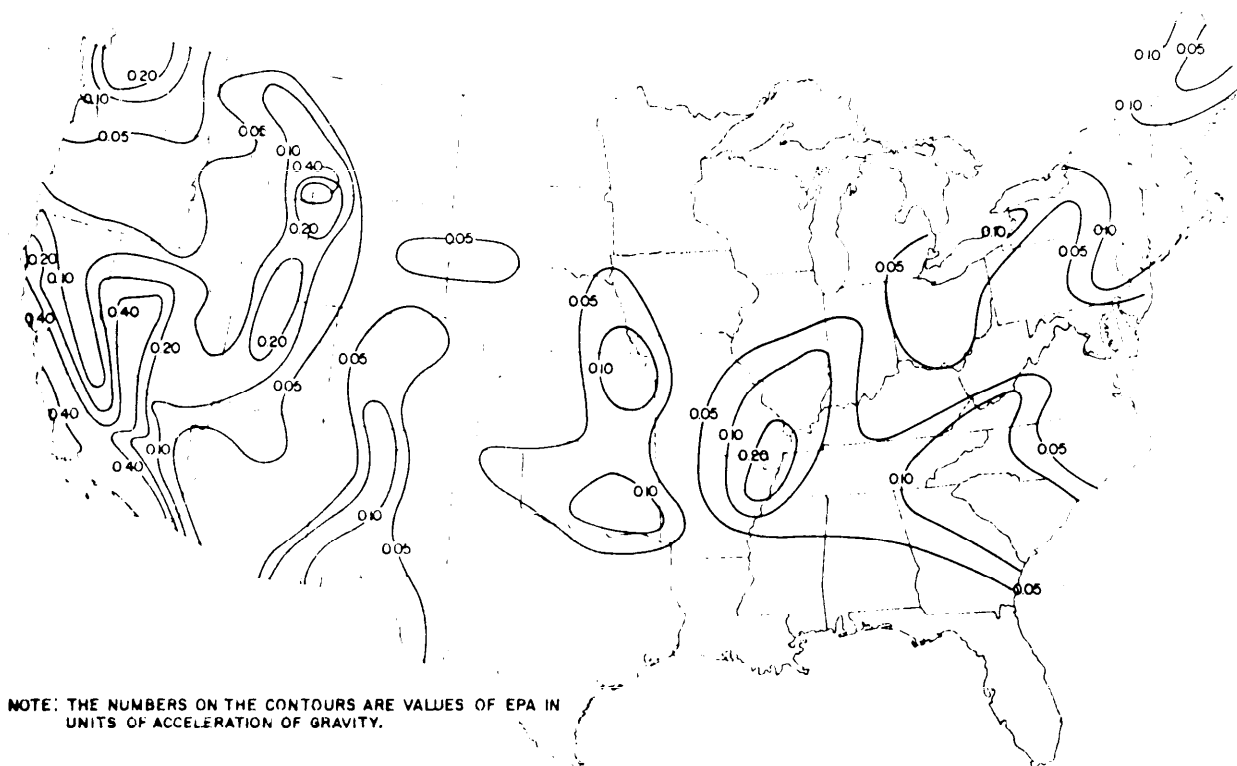
Section III. THE ATC3-06 METHOD

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.

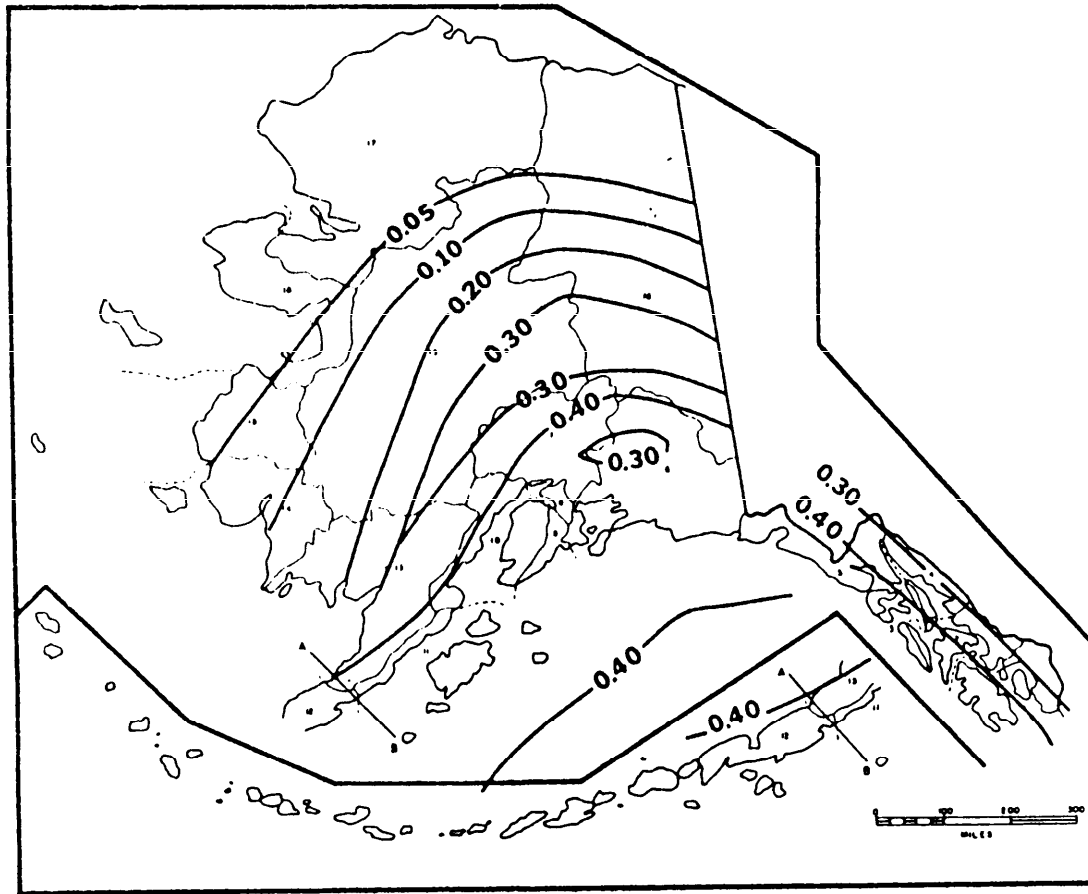
a. *Determination of site severity.* For a given

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.



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



ALASKA



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

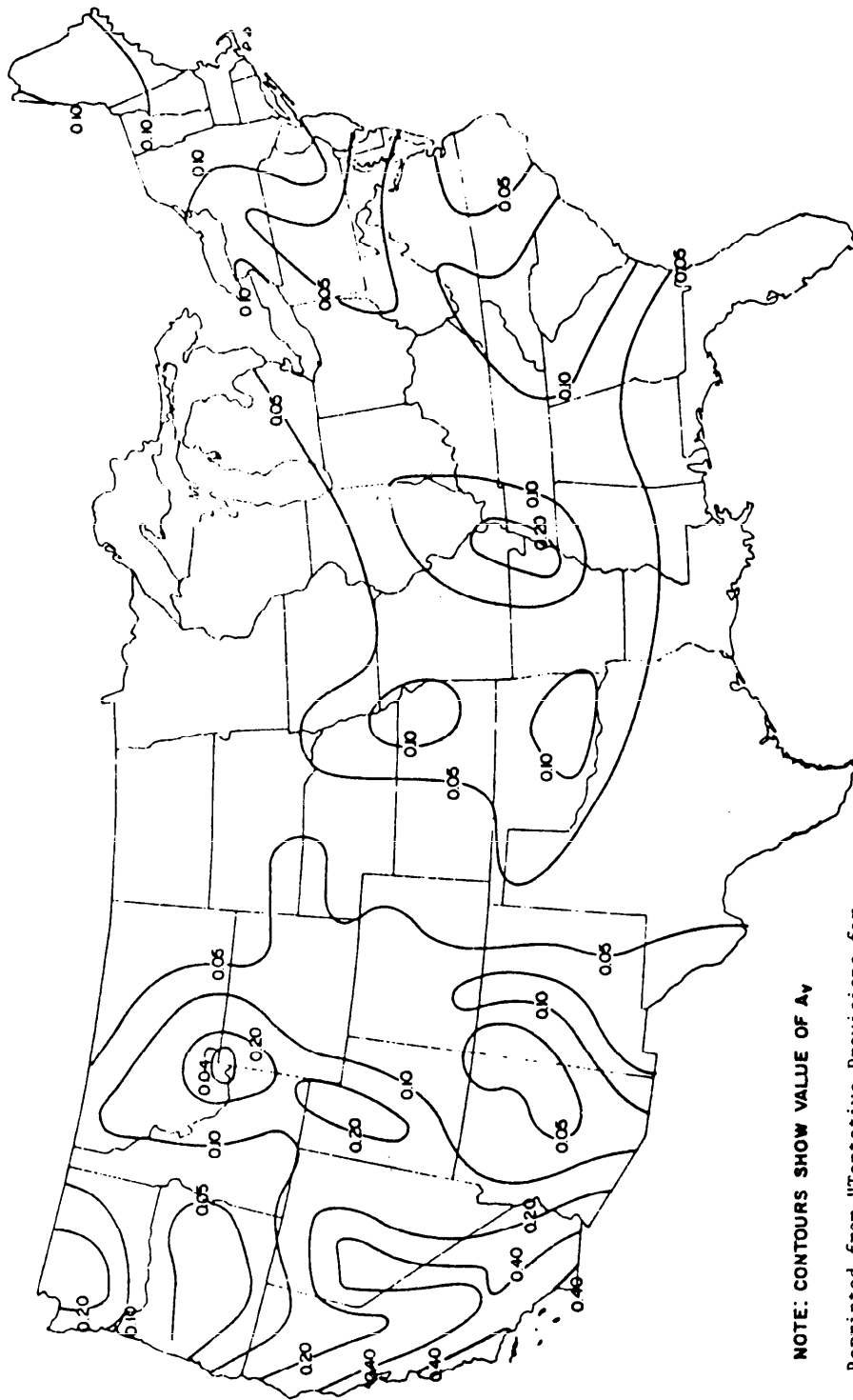
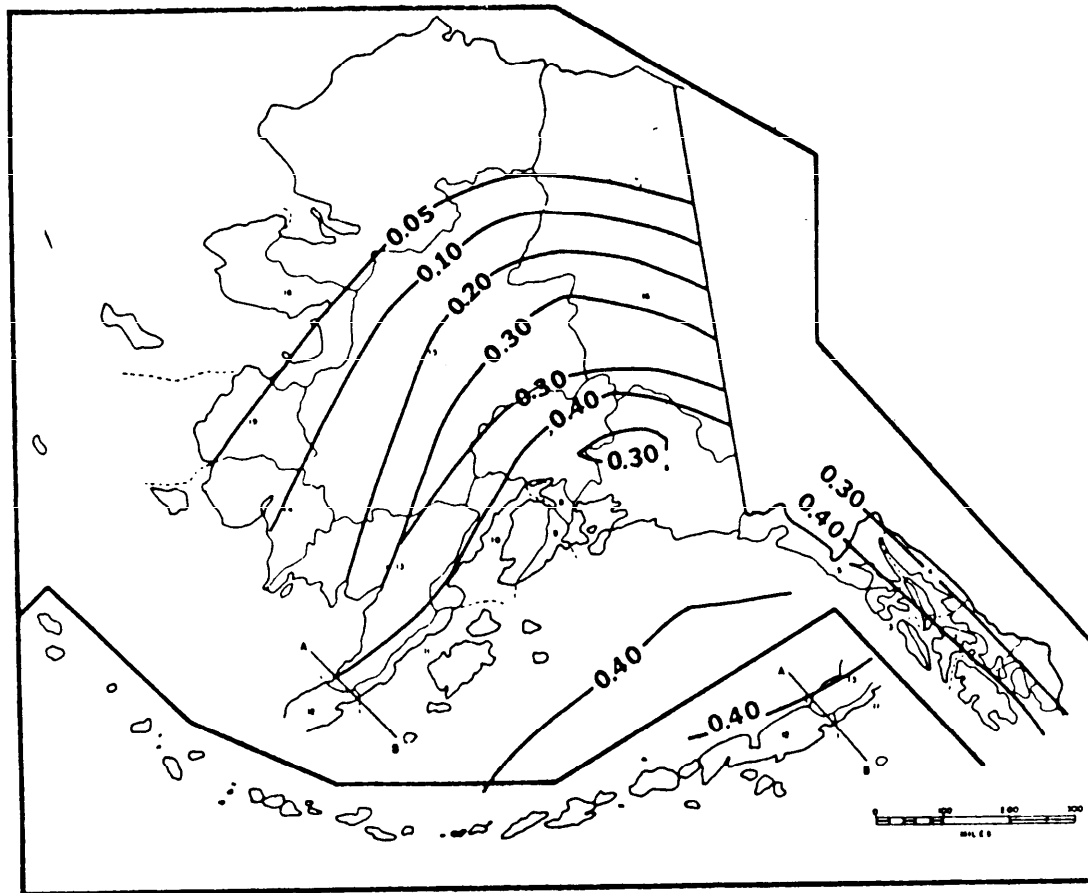
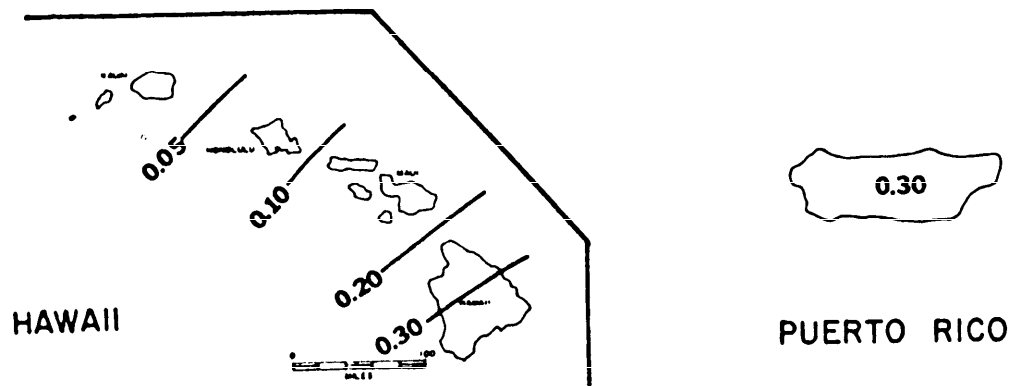


Figure 3-42. Contour map for effective peak velocity-related acceleration coefficient.



ALASKA

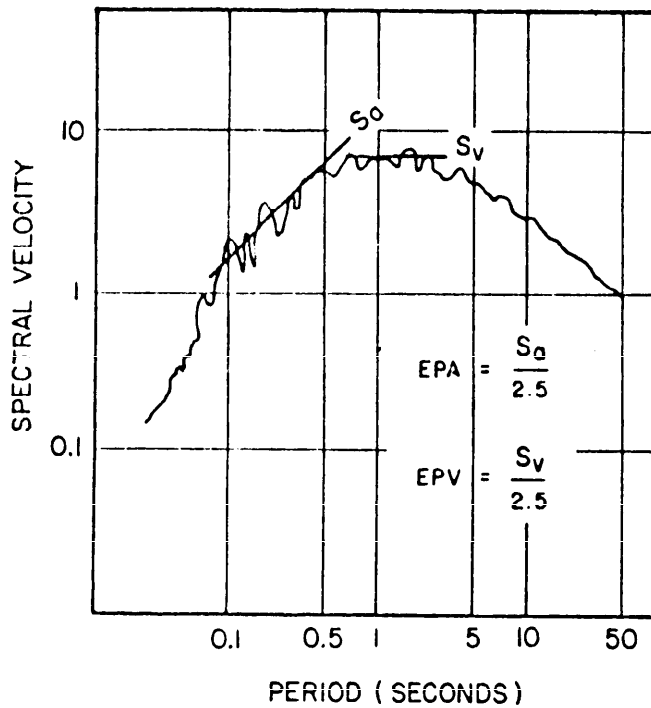


HAWAII

PUERTO RICO

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



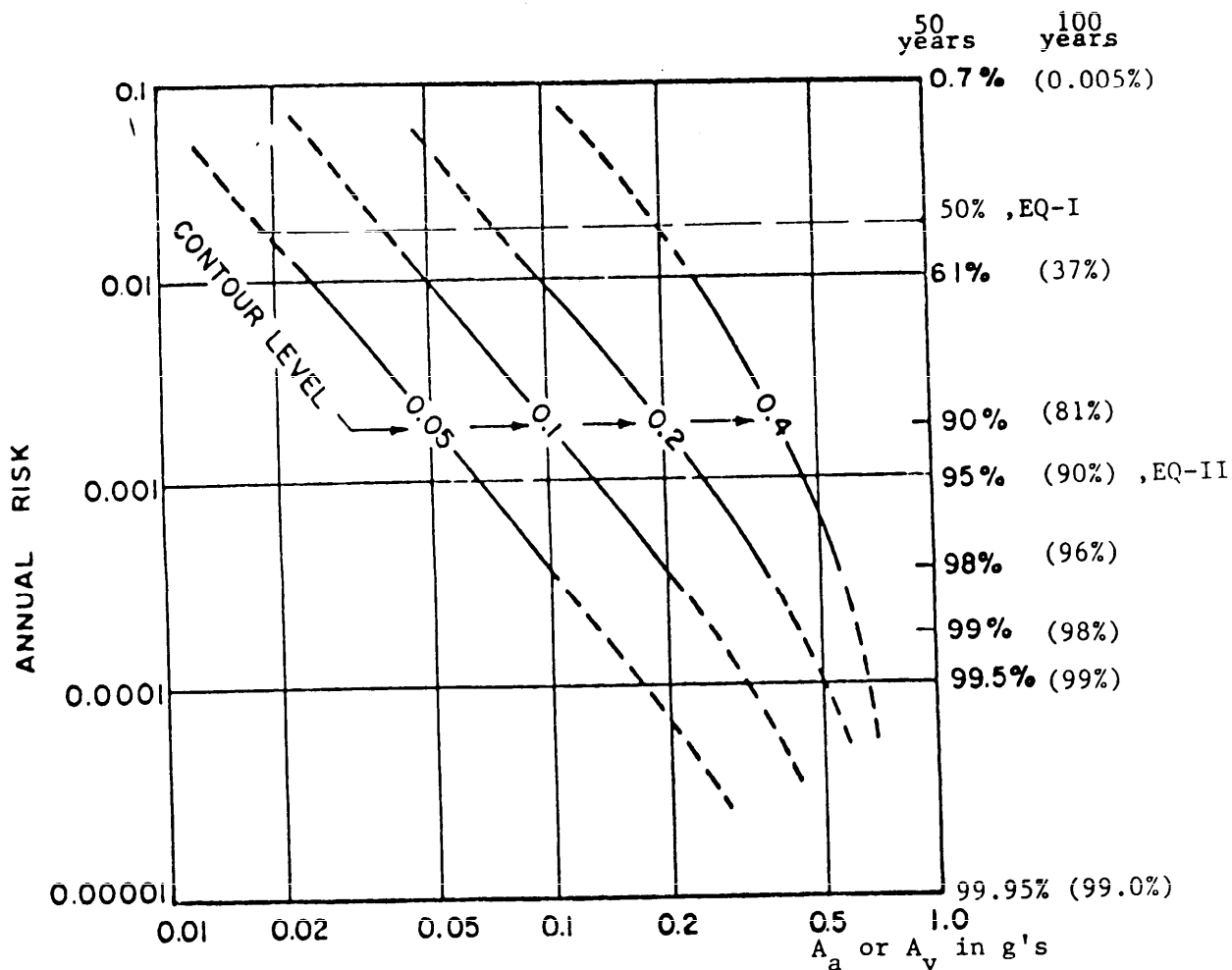
Map values of $A_a = \text{EPA}$ in g's

Map values of $A_v = \text{EPV}/30$ in g's, and EPV is in inches/sec.

A_v is the velocity related acceleration value.

Reprinted from "Tentative Provisions for the Development of Seismic Regulations for Buildings ATC 3-06," National Bureau of Standards, Special Publication 510, 1978.

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.

Reprinted from "Tentative Provisions for the Development of Seismic Regulations for Buildings ATC 3-06," National Bureau of Standards, Special Publication 510, 1978.

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.

Reprinted from "Tentative Provisions for the Development of Seismic Regulations for Buildings ATC 3-06," National Bureau of Standards, Special Publication 510, 1978.

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-2b of appendix C).

For $T \leq 4$ seconds:

$$S_a = \frac{1.22}{T} A_v S_i \text{ g's,} \quad (\text{eq 3-27})$$

(constant $S_v = 75A_v S_i$ in/
sec)

but always less or equal to

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

and

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

For $T > 4$ seconds:

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

$$\frac{150}{\pi} A_v S_i \text{ inches})$$

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

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

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.

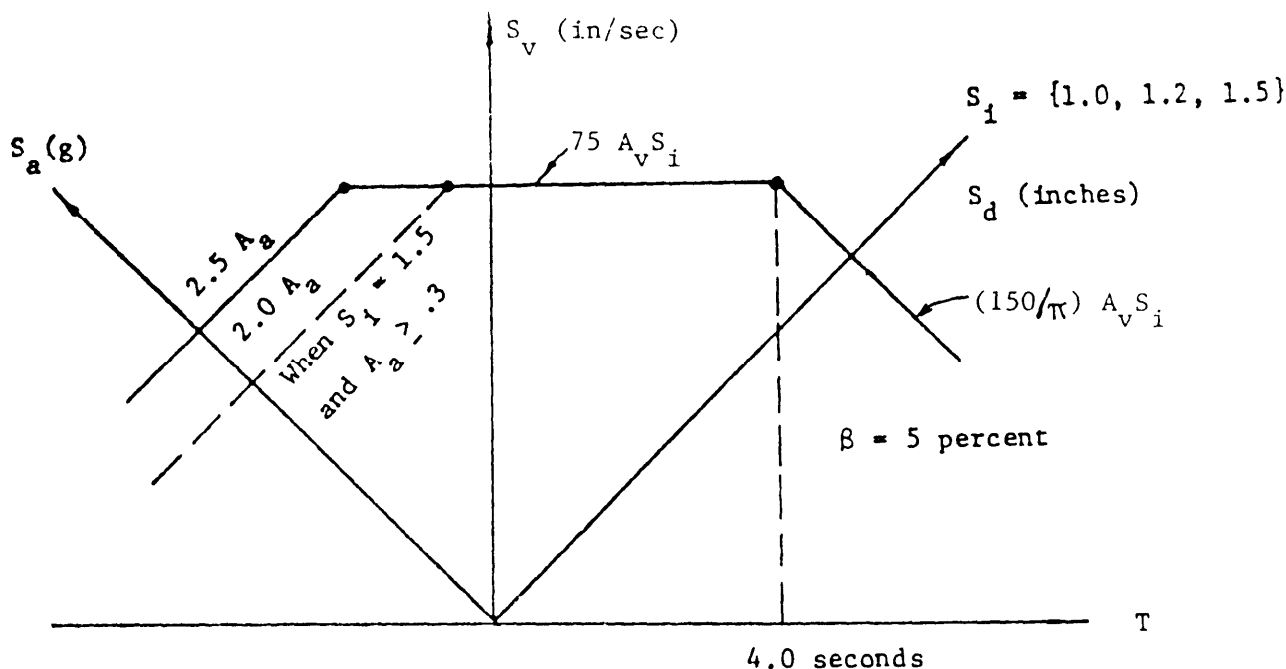
(1) The 5 percent damped EQ-I Spectrum is equal to the EQ-II spectrum multiplied by the ratio of the EQ-I to the EQ-II values given in table 3-4. Linear interpolation may be used for values between those given in this table.

(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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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.

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 am-

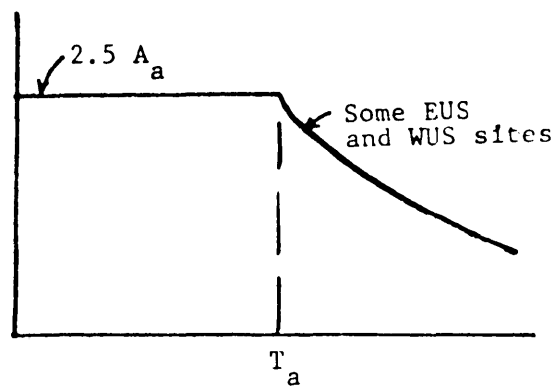
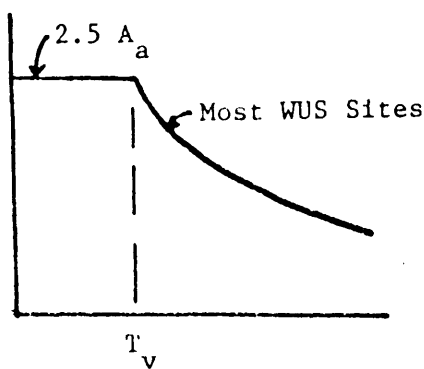
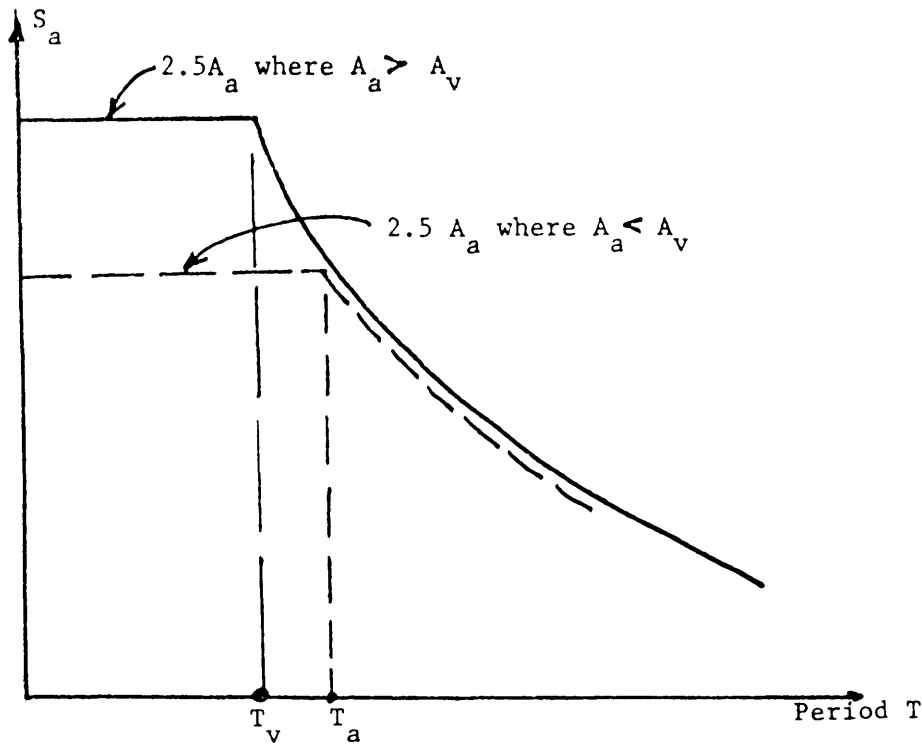
plitude of moderate frequency ground motion components, or a low attenuation of these components which is characteristic of the wave propagation in the EUS and in some regions of the WUS outside of California.

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,

$$\text{EQ-I, } A_a = 0.04, A_v = 0.06$$

$$\text{EQ-II, } A_a = 0.12, A_v = 0.18$$

(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

$$\begin{aligned} S_a &= (1.22/T)A_v S_i \text{ X Damping Adjustment} \\ &\quad \text{Factor,} \\ &= (1.22)(0.06)(1.2)(1.00)/T \\ &= (0.0878/T)g, \end{aligned}$$

but always less or equal to

$$\begin{aligned} S_a &= 2.5A_a \text{ X Damping Adjustment Factor} \\ &= 2.5(0.04)(1.00) \\ &= 0.10g \end{aligned}$$

EQ-II

$$\begin{aligned} S_a &= (1.22)(0.18)(1.2)(0.80)/T \\ &= (0.211/T)g \end{aligned}$$

but always less or equal to

$$\begin{aligned} S_a &= 2.5(0.12)(0.80) \\ &= 0.24g \end{aligned}$$

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:

$$\text{figure 3-46, } A_a = 0.40$$

$$\text{figure 3-48, } A_v = 0.40$$

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

$$\text{EQ-I, } A_a = 0.20, A_v = 0.20$$

$$\text{EQ-II, } A_a = 0.45, A_v = 0.45$$

(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

$$\begin{aligned} S_a &= (1.22/T)A_v S_i \text{ X Damping Adjustment} \\ &\quad \text{Factor} \\ &= (1.22)(0.20)(1.5)(1.00)/T \\ &= (0.366/T)g, \end{aligned}$$

but always less or equal to

$$\begin{aligned} S_a &= 2.0A_a \text{ X Damping Adjustment Factor} \\ &= 2.0(0.20)(1.00) \\ &= 0.40g \end{aligned}$$

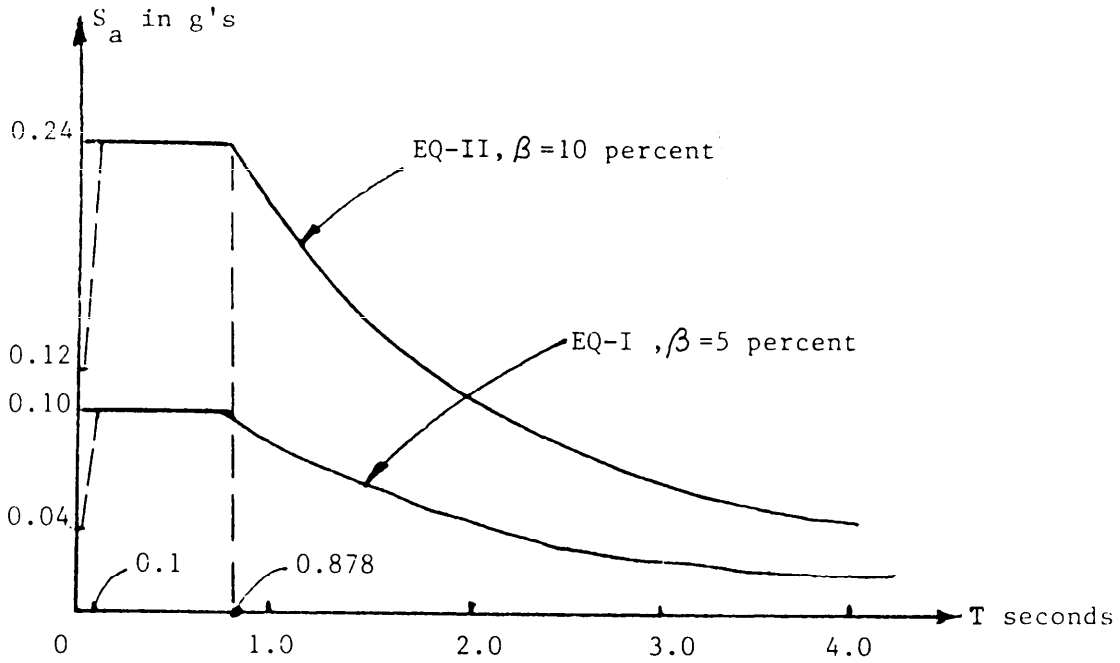
EQ-II

$$\begin{aligned} S_a &= (1.22)(.45)(1.5)(0.90)/T \\ &= (0.741/T)g \end{aligned}$$

but always less or equal to,

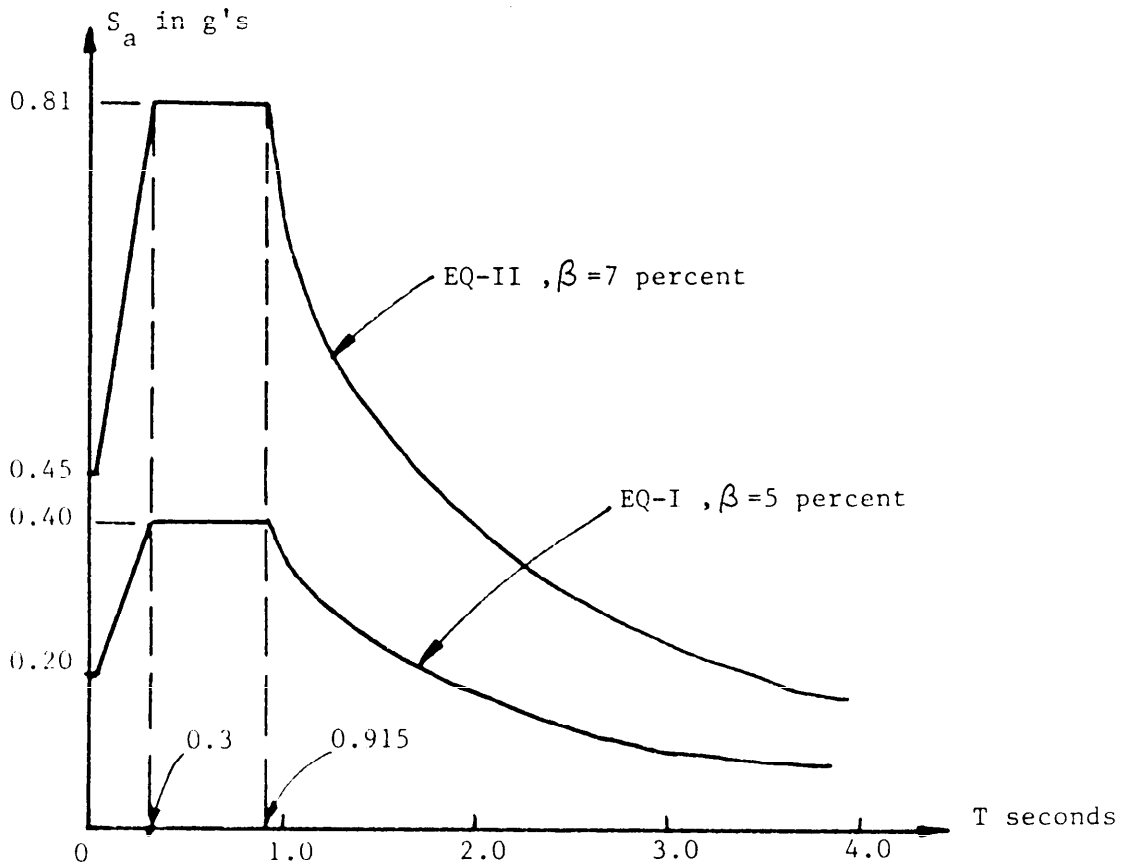
$$\begin{aligned} S_a &= (2.0)(0.45)(.90) \\ &= 0.81g \end{aligned}$$

(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 3-3b.

a. Essential facilities. Criteria set forth in this chapter have been developed primarily for the design of essential facilities, as classified in paragraph 1-1d, 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 1-1d, 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 1-1c), 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.

<u>Structural System</u>	<u>Elastic-Linear</u>	<u>Post Yield</u>
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 1-3a(2) for requirements), has unavoidable discontinuities in the vertical or horizontal planes (refer to para 1-3a(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 5-4b 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 5-4c(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:

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

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 $\sum(w/g)\phi^2$.

(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:

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

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:

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

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 4-3b (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:

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

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:

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

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., $\Delta_{xm} = \delta_{x+1m} - \delta_{xm}$).

(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 4-3e).

(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

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 5-4d(2) for clarification).

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

(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 5-4e(1) for clarification):

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

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

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.

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

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 5-4e(1) for clarification). These provisions parallel those of the Basic Design Manual, paragraph 4-4c(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 4-4c(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 4-3c 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., $f \text{ (calculated)} \leq 2.0 f \text{ (allowable)}$).

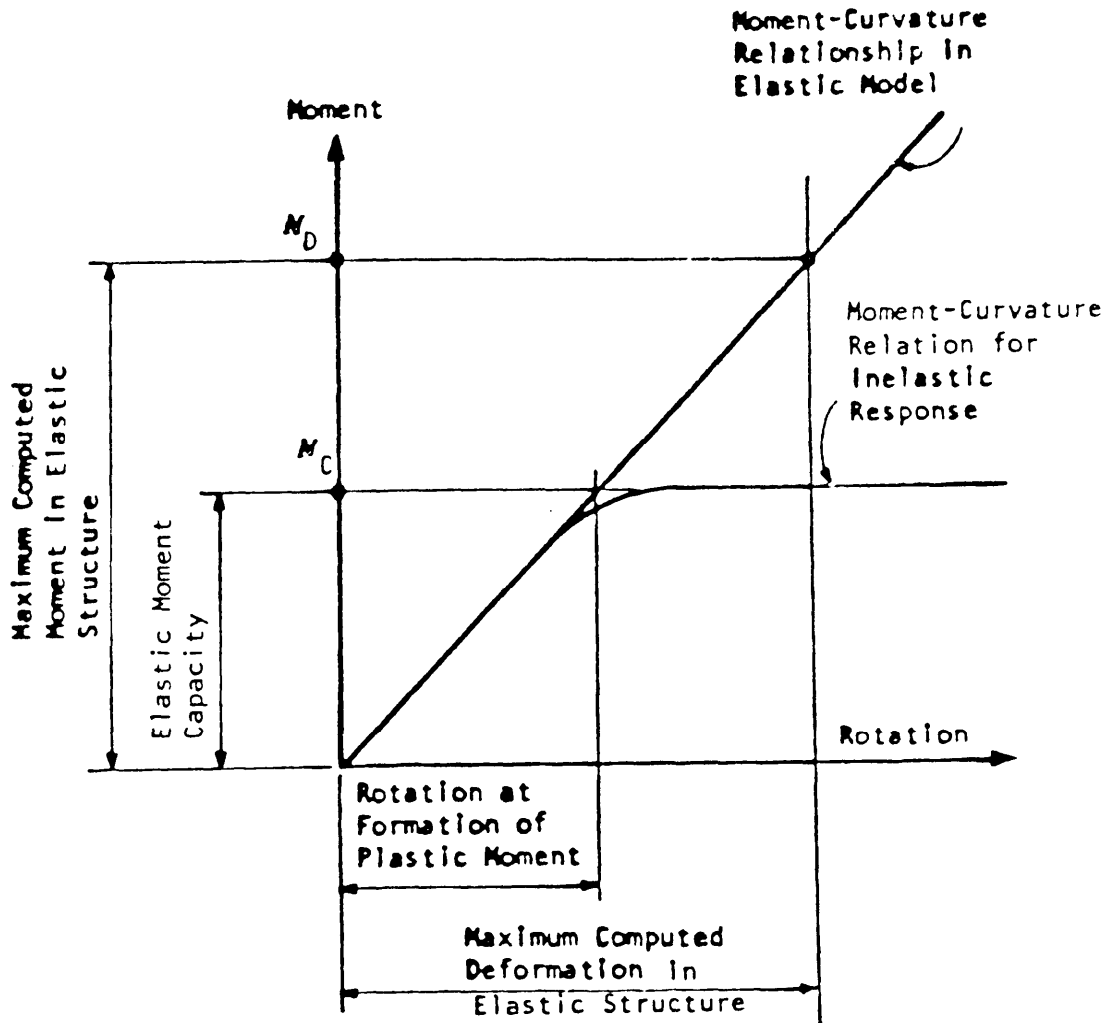
(5) *Connections.* All connections that do not develop the strength of the connecting mem-

bers 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 = $\sum A_s F_y$

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 1-1c). 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 4-3c) 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 4-4e. 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 Braced Frames	Beams	2.0	2.5	3.0
	Columns*	1.25	1.5	1.75
	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
Concrete DMRSF Concrete Walls	Connections	1.0	1.25	1.25
	Beams	2.0	2.5	3.0
	Columns*	1.25	1.5	1.75
	Shear Flexure	1.25 2.0	1.5 2.5	1.75 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:

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

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

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 facilities*. 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 (δ) com-

bined 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, 4-4d, and 4-8.

c. Capacities of buildings to resist demands of earthquakes. The ability of structures to resist the excessive accelerations and deforma-

tions 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 that 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 5-1a, 5-1b, and 5-1c, 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	=	<u>-150 k-ft</u>
 - b. Triple Seismic Forces:

DL + LL	=	-100 k-ft
Seismic	=	-150 k-ft
Total Beam Bending Moment	=	<u>-250 k-ft</u>
 - 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)	=	<u>-13 k-ft</u>
 - b. Triple Seismic Forces:

0.9 DL	=	-63 k-ft
Seismic	=	150 k-ft
Net Moment (Reverses to positive bending moment)	=	<u>87 k-ft</u>
 - 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.

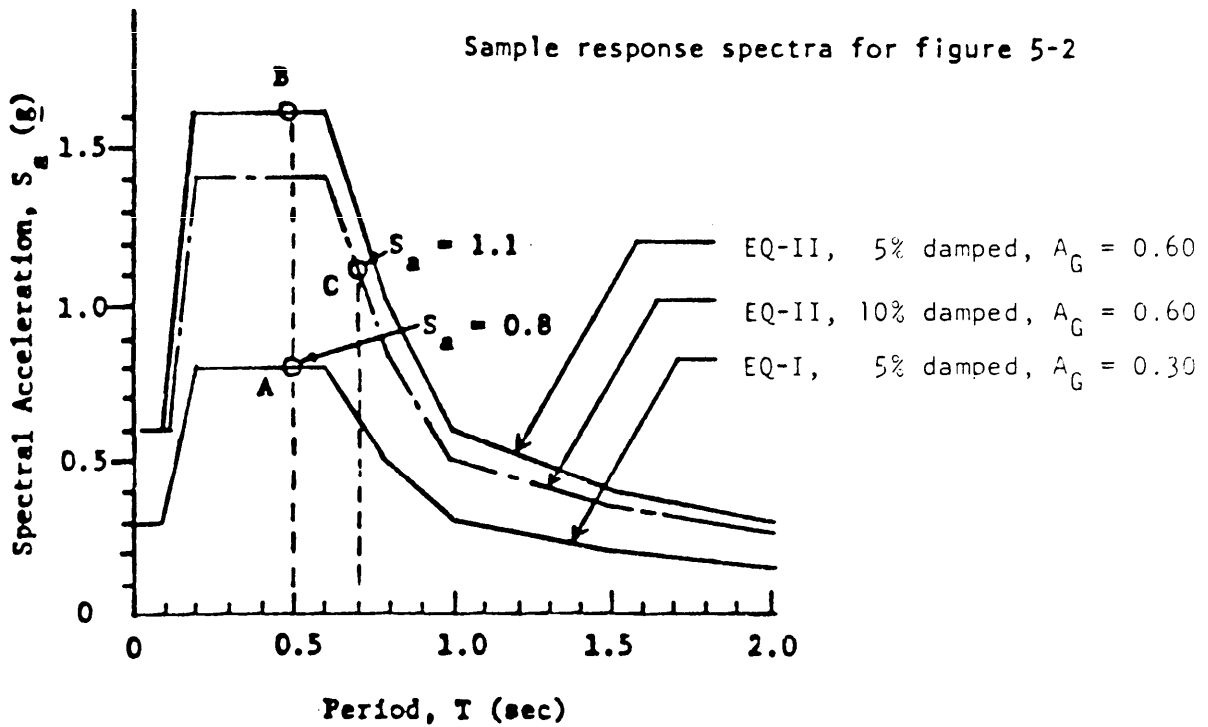
Dynamic Structural Characteristics. Because of the relationships between dynamic characteristics of structures and the dynamic properties of seismic ground motion, the effective forces applied to the structure are not directly proportional to the peak ground acceleration of the earthquake. The periods 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 ($S_a = 0.8 \text{ g}$), 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 ($S_a = 1.6 \text{ g}$) 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.

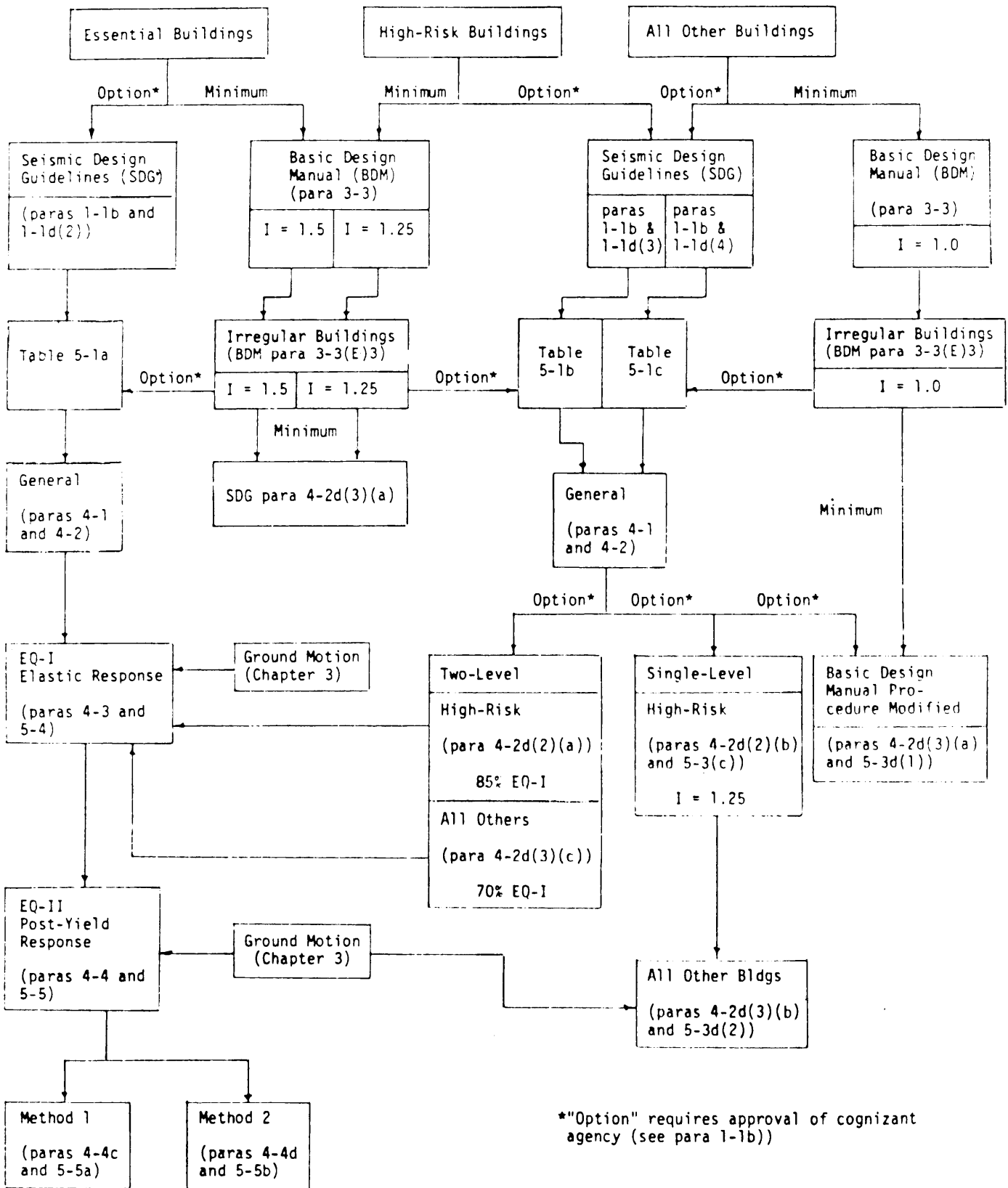
Figure 5-2. Dynamic structural characteristics.



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.



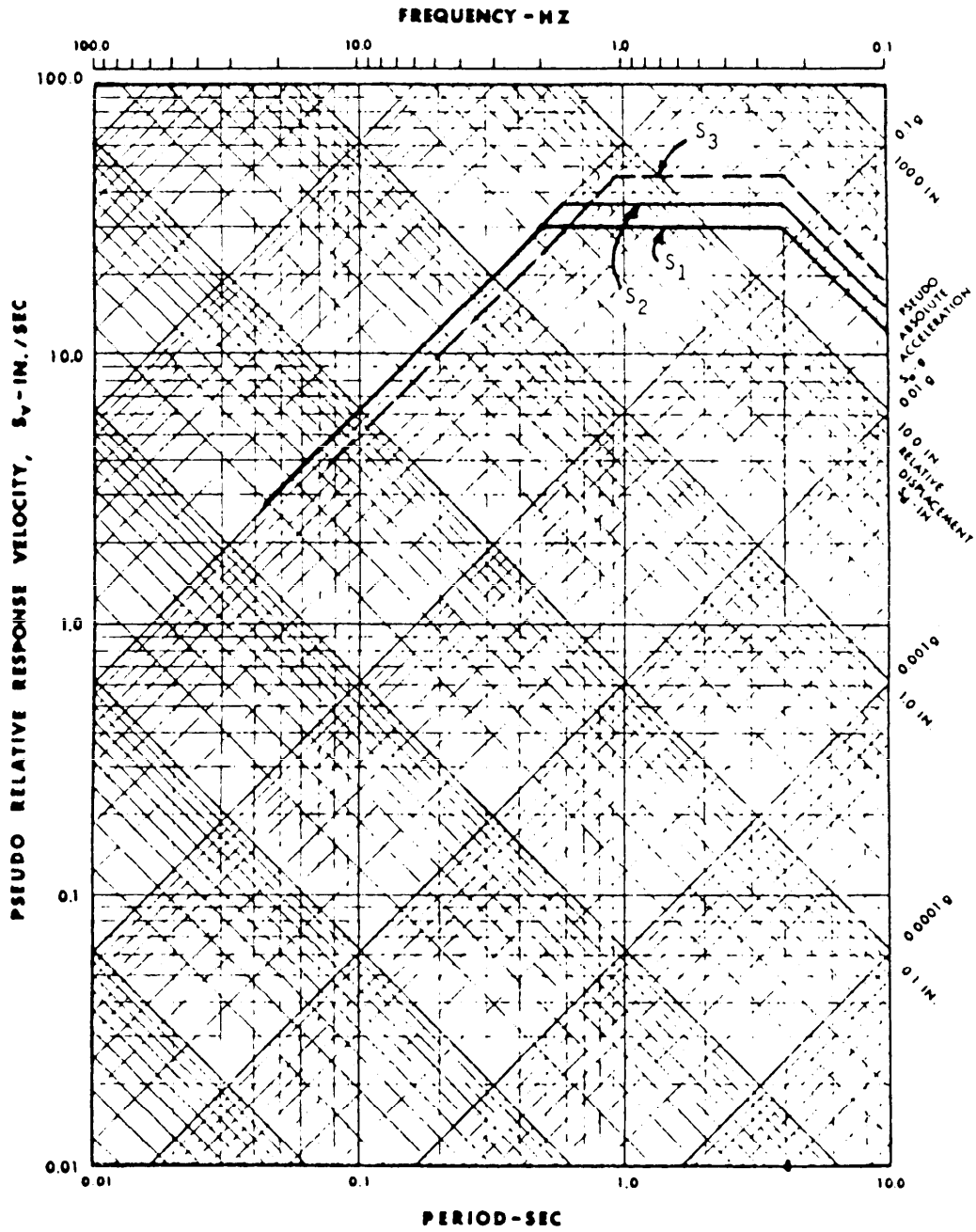
**"Option" requires approval of cognizant agency (see para 1-1b))

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

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

	<u>Requirements</u>	<u>Procedure</u>
Classification of building	4-1a	
General requirements	4-2a	
Dynamic Analysis procedure	4-2d(1)	5-3b
<u>EQ-I</u>		
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
<u>EQ-II</u>		
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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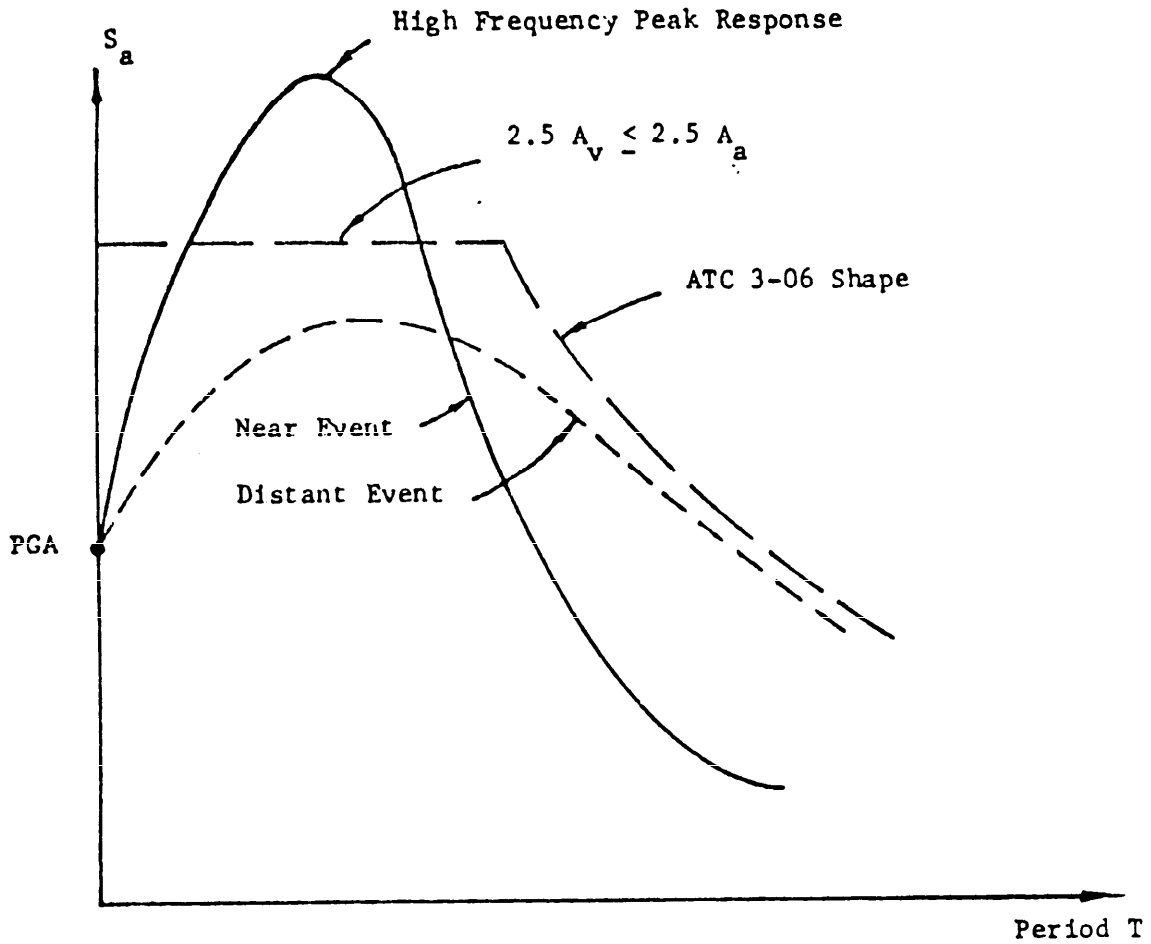
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Figure 3-47. EQ-II spectra for $A_a = A_v = 0.40$, and $\beta = 5\%$.

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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Figure 3-48. Effective spectral envelope.

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

	<u>Requirements</u>	<u>Procedure</u>
Classification of building	4-1b	
General requirements	4-2a	
Dynamic analysis procedure	4-2d(2)	5-3
 <u>Two Level Approach</u>	 4-2d(2)(a)	
Same as essential facilities (Table 5-1a) except the following:		
<u>EQ-I</u>		
Response spectrur	85% of EQ-I, 4-2d(2)(a)	
Drift limits	Increase 40%, 4-3e(7)	
<u>EQ-II</u>		
Drift limits	Increase 50%, 4-4e(2)	
Inelastic demand ratio	High-risk column of table 4-2	
 <u>Single Level Design</u>		
Same as <u>Basic Design Manual Procedure with modified seismic force distribution and Single level design for EQ-I with minimum story shear requirements</u> 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.

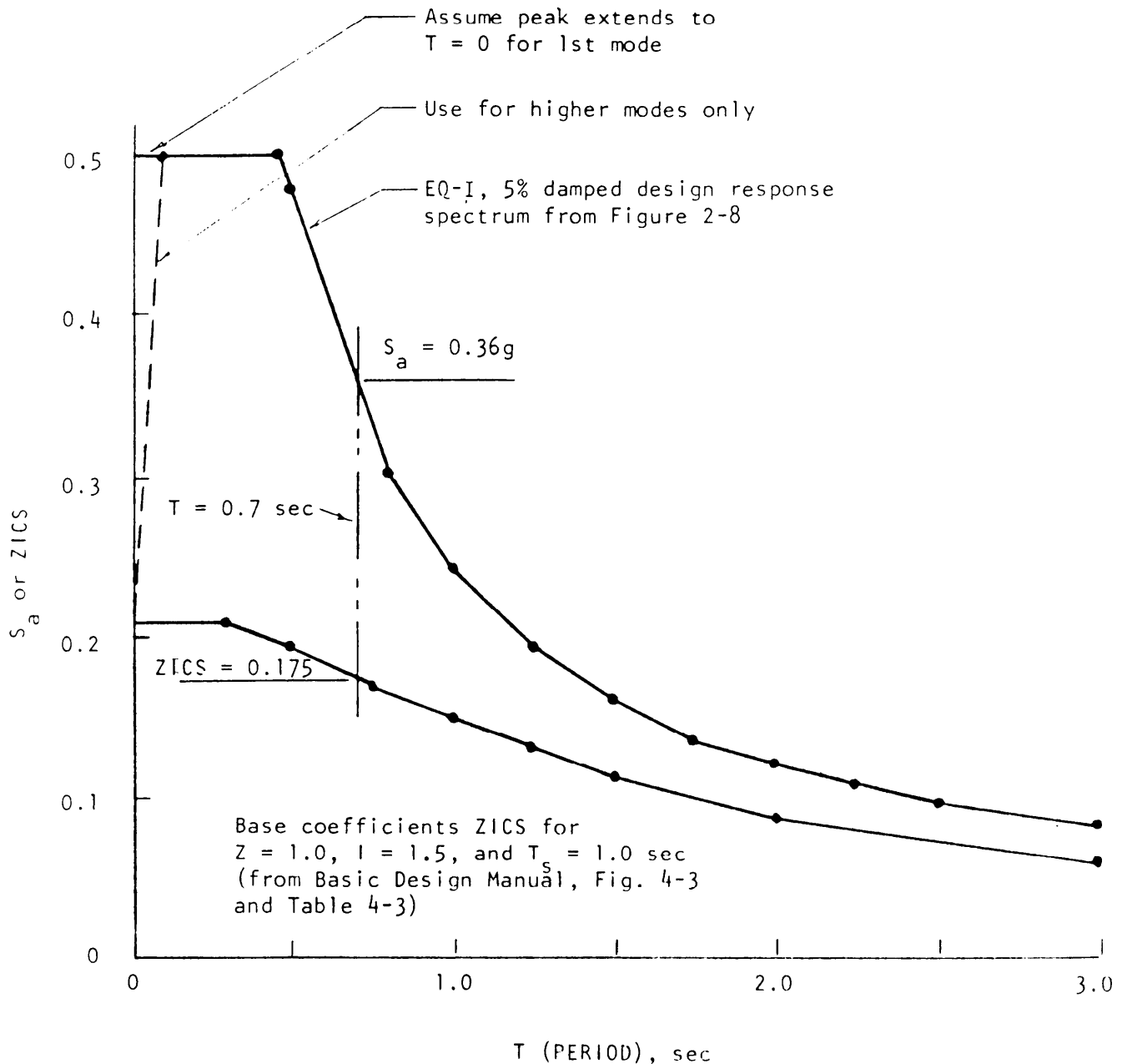
	<u>Requirements</u>	<u>Procedure</u>
Classification of building	4-1c	
General requirements	4-2a	
Dynamic analysis procedure	4-2d(3)	
<u>Basic Design Manual Procedure with modified seismic force distribution</u>		
General	4-2d(3)(a)	5-3d
Response spectrum EQ-I	4-3b	
Select structural system	Basic Design 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	
<u>Single level design for EQ-I with minimum story shear requirements</u>		
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
<u>Two level approach</u>		
General	4-2d(3)(c)	5-3d(3)
Same as essential facilities except for the following:		
<u>EQ-I</u>		
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)
<u>EQ-II</u>		
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 $C \times S$ 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 $0.1N = 0.7$ 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 proportionately 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 4-3c(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 4-3d and 5-4j 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 4-2d(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 4-2d(2). The choice will generally depend on the seismic severity of the site, type of build-

ing, 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 4-2d(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 4-3e(7) and 4-4e(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 4-2d(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 4-2d(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 4-2d(3)(a) and (b) are both single-level design procedures. These procedures will generally be sufficient for most buildings. Paragraph 4-2d(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 4-2d(3)(a)). This procedure uses the modal analysis method to determine the distribution of lateral forces along the height of the structure in lieu 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, $V_T = \sqrt{\sum V_m^2}$. If data to develop the EQ-I response spectrum are not available, the equations in paragraph 3-8c 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 descriptions 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:

$$R_v = ZIKCSW / \sqrt{\sum V_m^2} \quad (\text{eq 5-1})$$

(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 val-

ues 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 ($1.5 \div 1.4 = 1.07$).

(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 5-3c 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 4-2d(3) (c)). The effect of this reduction is similar to that indicated above for high-risk facilities (para 5-3c(1) (a)).

(b) The drift limits for EQ-I (0.007) and EQ-II (0.015) are less severe (para 4-3e(7) and 4-4e(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 5-3a. 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:

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

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:

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

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	-0.852	-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	-0.068	-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)				$\Delta a/hs$
	1	2	3	SRSS	1	2	3	SRSS	1	2	3	SRSS	
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 2-5c 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 5-4d, 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 masonry 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 sideways 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 5-4a. 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) *Accidental 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 accidental 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 5-2d(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 compatibility.* 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 5-4b 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 5-4c(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 4-3f. 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:

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

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

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 non-structural elements in paragraph 6-4. For its use in the EQ-I analysis, refer to paragraph 5-5b.

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 4-3e(7)(a). The maximum story displacements are required for the criteria for building separations in paragraph 4-3e(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:

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

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 4-3d 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 5-3d(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 5-3a, 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 5-4a. 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 5-4b 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 distribu-

tion 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 5-4i. 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.

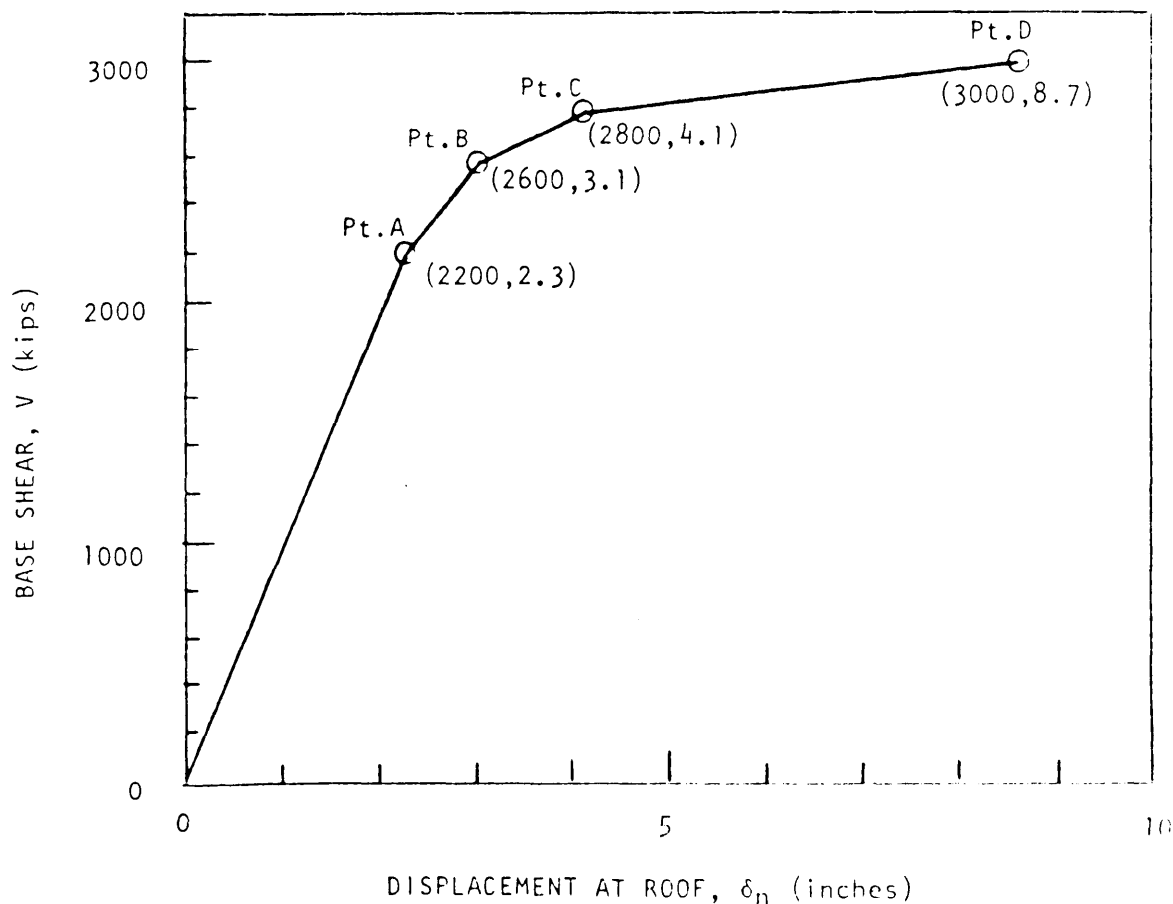
(f) Unusual distributions of inelastic demand ratios. This is a more general case of paragraphs (d) and (e), above. This provision is used to check the efficiency of the overall lateral-force-resisting system. If a limited number of structural elements have large *inelastic demand ratios* and the remainder 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 the

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

$PF_N = (\sum m\phi) (\phi_N) / (\sum m\phi^2)$, modal roof participation factor (eq. 4-1)

$\alpha = (\sum m\phi)^2 / (\sum m) (\sum m\phi^2)$, effective modal weight (eq. 4-2)

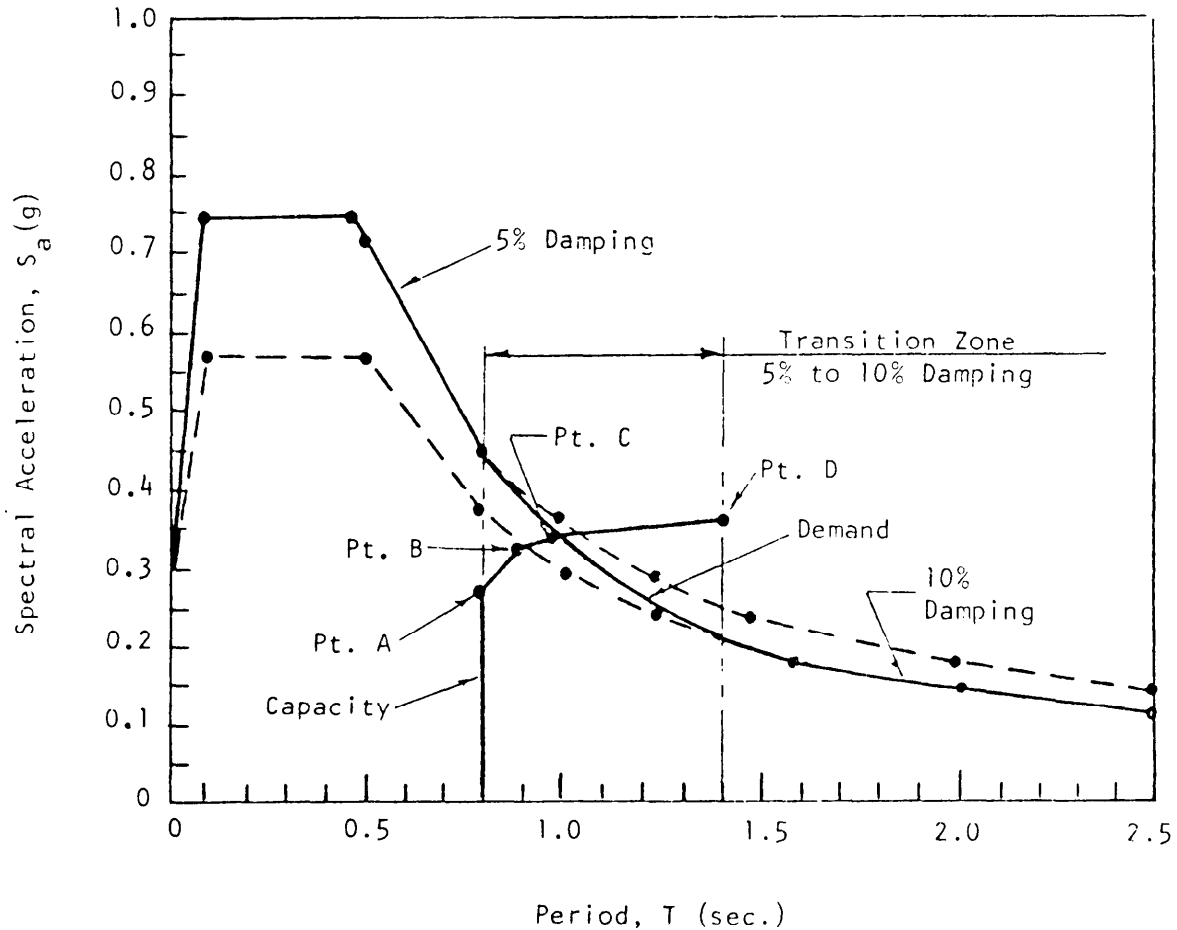
S_a = Spectral acceleration = $V/W \div \alpha$ (eq. 4-4)

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)

$\sum m\phi$ = 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 multi-mode 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 ca-

capacity curve. The following procedure can be set-up 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, interaction diagrams at $\phi = 1.0$ 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.

$$\text{Net earthquake capacity} = EC - D - 0.25L \quad (\text{eq 5-7})$$

$$\text{Net earthquake capacity} = EC + D \quad (\text{eq 5-8})$$

(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 5-4e(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 $\delta_n = 2.3$ 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 $\delta_n = 3.1$ 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 re-

duced 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 $\delta_n = 8.7$ inches in table 5-4 and figure 5-5.

(h) Determine 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.

$$\begin{aligned} \delta_n &= PF_n S_a (T/2\pi)^2 g \\ &= 1.28 \times 0.35 (1/2\pi)^2 386 = 4.38 \text{ inches} \end{aligned}$$

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 4-4e(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 5-5a. 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-

terminated 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 4-4e(2). Methods of calculating the displacements are described in paragraphs 5-5a and 5-5b. 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 5-5d.

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:

$$a_{xm} = PF_{xm}S_{am} \quad (\text{eq 6-1})$$

where:

$$a_{xm} = \text{modal story acceleration at level } x \text{ for mode } m.$$

$$PF_{xm} = \text{modal participation factor as determined by equation 4-1.}$$

$$S_{am} = \text{spectral acceleration for mode } m.$$

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.

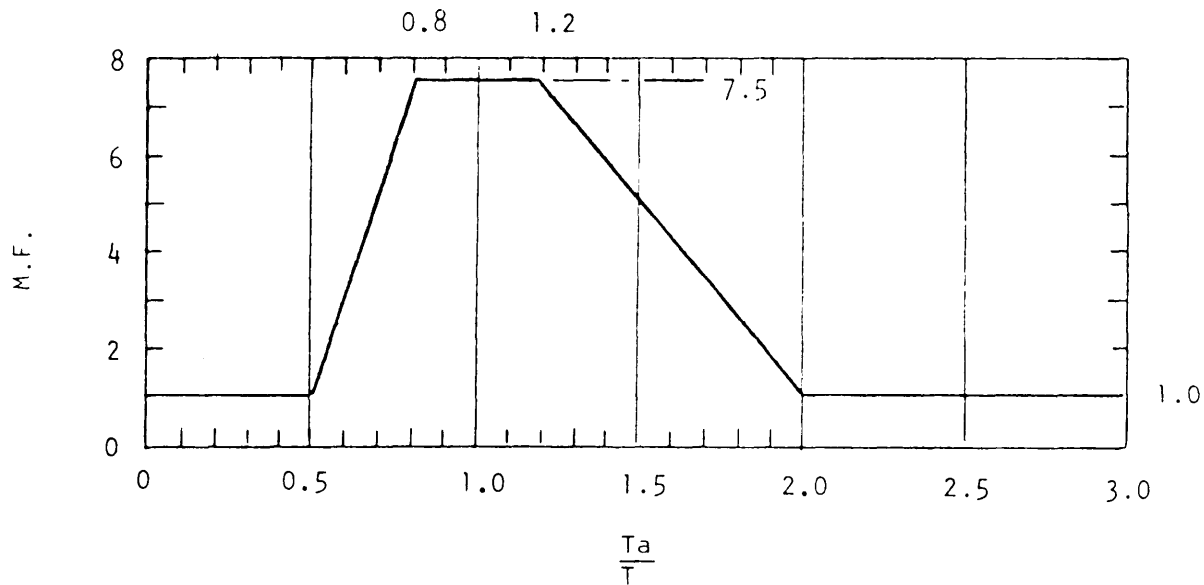
$$(a_x)_{\max} = \sqrt{\sum a_{xm}^2} \quad (\text{eq 6-2})$$

(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_{ra} (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:

$$T_a/T_m = T_a/T \tag{eq 6-3}$$

$$S_{fa} = a_{xm} \text{ (M.F.)} \tag{eq 6-4}$$

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., $T_a < 0.05$ sec) will be designed to resist the forces due to the maximum floor acceleration in accordance with the equation 6-5:

$$F_p = (a_x)_{max} W_p \tag{eq 6-5}$$

where $(a_x)_{max}$ is determined from paragraph 6-3b 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:

$$F_p = S_{fax} W_p \tag{eq 6-6}$$

where S_{fax} is the design spectral acceleration, S_{fa} , at floor x as defined in paragraph 6-3c 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 6-3c 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 6-3d.

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

$$T_a = T_m (T_a/T) = 2.0 (T_a/T)$$

$$S_{fa} = a_{xm} (\text{M.F.}) = 0.12 (\text{M.F.})$$

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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)

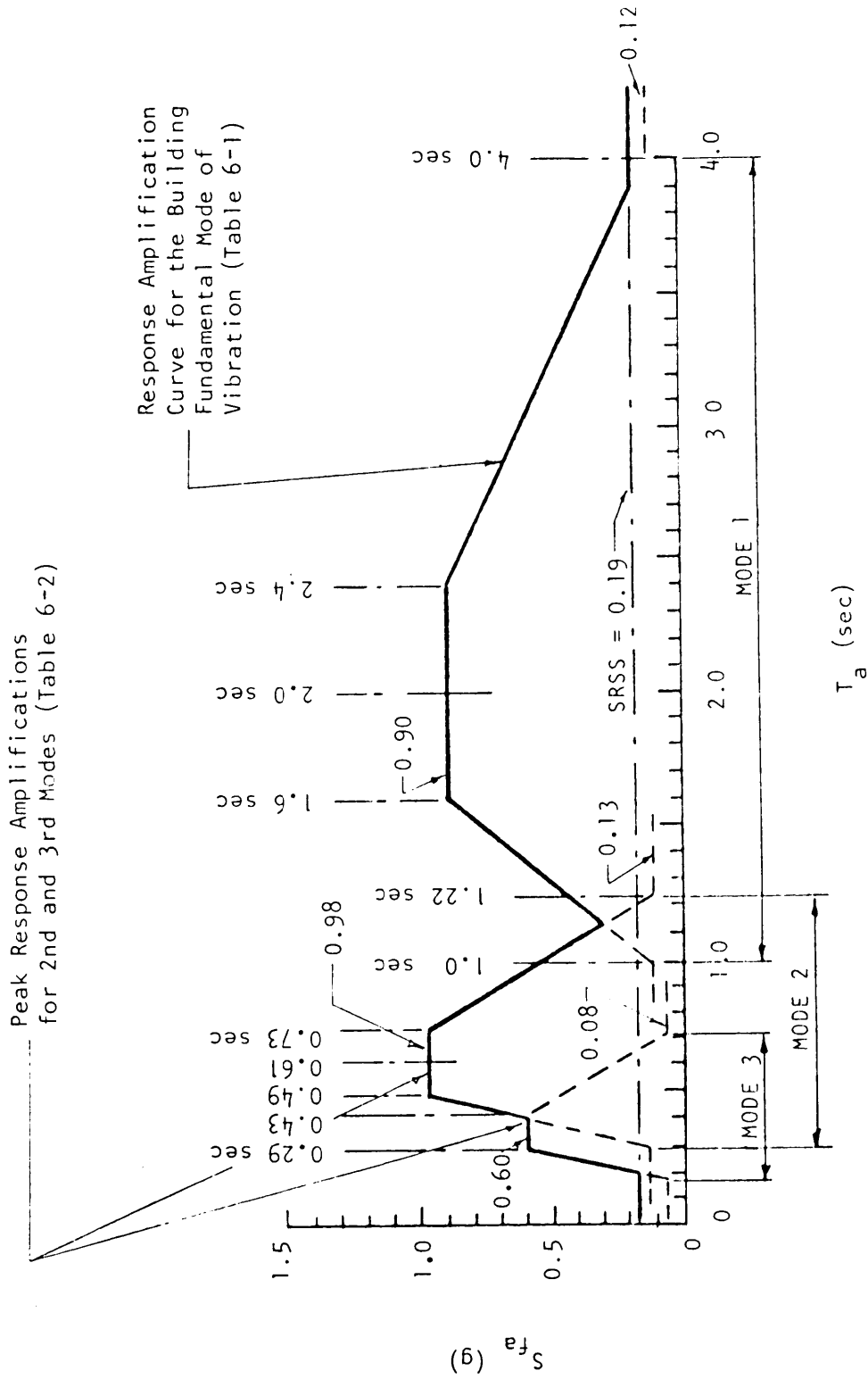
S_{am} from response spectrum for building period T_m

PF_{xm} from building analysis (eq.4-1)

$a_{xm} = PF_{xm} S_{am}$ is modal 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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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.

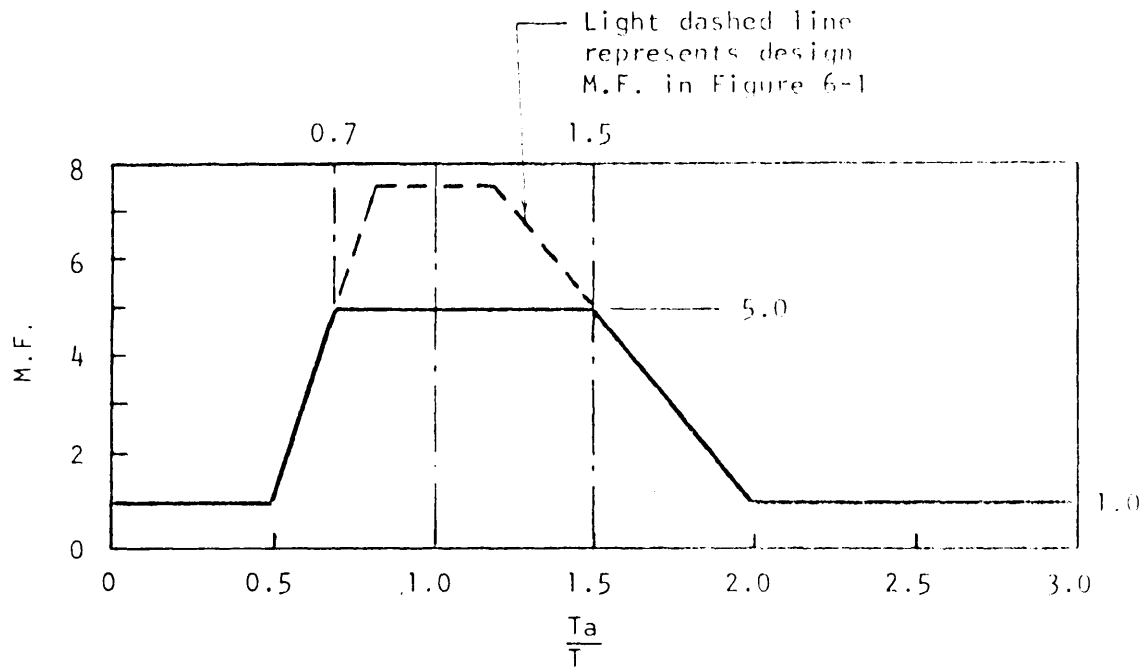
Architectural elements must: (1) safely resist horizontal forces equal to the design accelerations times their own weight; and (2) be capable of conforming (accommodating) to the lateral deflections that they will be subjected to during lateral deformation of the building in which they are located. The 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 10-7a), 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 4-7d). Any glass used within exitways shall be tempered and its frame shall be designed to allow deformations (Basic Design Manual, chap 9, para 9-4e). 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 caused 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 ₂ O		
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 6-7f, 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 6-7f.

(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 adequately 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 occurs 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 12-5b and 12-6c). 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 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, lateral 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 lateral clearance is provided between adjacent piping, ducts, hangers, or other elements. Walls, including nonbearing gypsum board partitions, may be considered transverse bracing for ducts that pass through them. Ducts may be grouped in a lateral 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 1/2-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 9-4b). 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 partitions 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 9-4a).

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 9-4a). 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 7-2a and will not be permitted the exception of paragraph 7-2b. 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 11-3b, 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 accel-

eration 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.