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Response Spectra and Seismic Analysis for Concrete Hydraulic Structures

ENGINEER MANUAL

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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

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Engineering and Design
RESPONSE SPECTRA AND SEISMIC ANALYSIS FOR CONCRETE HYDRAULIC STRUCTURES

1. Purpose. This manual describes the development and use of response spectra for the seismic analysis of concrete hydraulic structures. The manual provides guidance regarding how earthquake ground motions are characterized as design response spectra and how they are then used in the process of seismic structural analysis and design. The manual is intended to be an introduction to the seismic analysis of concrete hydraulic structures. More detailed seismic guidance on specific types of hydraulic structures will be covered in engineer manuals and technical letters on those structures.

2. Applicability. This manual applies to all USACE Commands having responsibilities for the design of civil works projects.

3. Scope of Manual. Chapter 1 provides an overview of the seismic assessment process for hydraulic structures and the responsibilities of the project team involved in the process, and also briefly summarizes the methodologies that are presented in Chapters 2 and 3. In Chapter 2, methodology for seismic analysis of hydraulic structures is discussed, including general concepts, design criteria, structural modeling, and analysis and interpretation of results. Chapter 3 describes methodology for developing the earthquake ground motion inputs for the seismic analysis of hydraulic structures. Emphasis is on developing response spectra of ground motions, but less detailed guidance is also provided for developing acceleration time-histories.

4. Distribution Statement. Approved for public release; distribution is unlimited.

FOR THE COMMANDER:

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Table of Contents

Chapter 1
Introduction

1-1. Purpose

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1-4. References

References are listed in Appendix A.

1-5. Responsibilities of Project Team

The development and use of earthquake ground motion inputs for seismic analysis of hydraulic structures require the close collaboration of a project team that includes the principal design engineer, seismic structural analyst, materials engineer, and geotechnical specialists. The principal design engineer is the leader of the project team and has overall responsibility for the design. The seismic structural analyst plans, executes, and evaluates the results of seismic analyses of the structure for earthquake ground motions for the design earthquakes. The materials engineer characterizes the material properties of the structure. The geotechnical specialists conduct evaluations to define the design earthquakes and input ground motions and also characterize the properties of the soils or rock foundation for the structure. Any potential for seismically induced failure of the foundation is evaluated by the geotechnical specialists. The geotechnical evaluation team typically involves the participation of geologists, seismologists, and geotechnical engineers.

1-6. Overview of Seismic Assessment

The overall process of seismic assessment of concrete hydraulic structures consists of the following steps: establishment of earthquake design criteria, development of design earthquakes and characterization of earthquake ground motions, establishment of analysis procedures, development of structural models, prediction of earthquake response of the structure, and interpretation and evaluation of the results. The following paragraphs present a brief description of each step, the objectives, and the personnel needed to accomplish the tasks.

a. Establishment of earthquake design criteria. At the outset, it is essential that the lead members of the project team (principal design engineer, seismic structural analyst, materials engineer, and lead geotechnical specialist) have a common understanding of the definitions of the project operating basis earthquake (OBE) and maximum design earthquake (MDE). Structure performance criteria for each design earthquake should also be mutually understood. Having this understanding, the geotechnical team can then proceed to develop an overall plan for developing design earthquakes and associated design response spectra and acceleration time-histories, while the structural team begins establishing conceptual designs and analysis and design methods leading to sound earthquake-resistant design or safety evaluation.

b. Development of design earthquakes and characterization of earthquake ground motions.

(1) Assessing earthquake potential. The project geologist and seismologist must initially develop an understanding of the seismic environment of the site region. The seismic environment includes the regional geology, regional tectonic processes and stress conditions leading to earthquakes, regional seismic history, locations and geometries of earthquake sources (faults or source areas), and the type of faulting (strike slip, reverse, or normal faulting). Analysis of remote imagery and field studies to identify active faults may be required during this step. Next, maximum earthquake sizes of the identified significant seismic sources must be estimated (preferably in terms of magnitude, but in some cases in terms of epicentral Modified Mercalli intensity). Earthquake recurrence relationships (i.e., the frequency of occurrence of earthquakes of different sizes) must also be established for the significant seismic sources.

(2) Determining earthquake ground motions. After the geologist and seismologist have characterized the seismic sources, the geotechnical engineer and/or strong-motion seismologist members of the geotechnical team can then proceed to develop the design (OBE and MDE) ground motions, which should include response spectra and, if needed, acceleration time-histories as specified by the principal design engineer. The design ground motions should be based on deterministic and probabilistic assessments of ground motions. These design ground motions should be reviewed and approved by the principal design engineer.

c. Establishment of analysis procedures.

(1) Basic entities of analysis procedures. The establishment of analysis procedures is an important aspect of the structural design and safety evaluation of hydraulic structures subjected to earthquake excitation. The choice of analysis procedures may influence the scope and nature of the seismic input characterization, design procedures, specification of material properties, and evaluation procedures of the results. The basic entities of analysis procedures described in this manual are as follows: specification of the form and point of application of seismic input for structural analysis, selection of method of analysis and design, specification of material properties and damping, and establishment of evaluation procedures.

(2) Formulation of analysis procedures. The analysis procedures and the degree of sophistication required in the related topics should be established by the principal design engineer. In formulating rational structural analysis procedures, the principal design engineer must consult with experienced seismic structural, materials, and geotechnical specialists to specify the various design and analysis parameters as well as the type of seismic analysis required. The seismic structural specialist should review the completed design criteria for adequacy and in the case of major projects may work directly with the engineering seismologists and the geotechnical engineers in developing the seismic input. The physical properties of the construction materials and the foundation supporting the structure are determined in consultation with the materials, geotechnical engineer, and the engineering geologist (for the rock foundation).

d. Development of structural models. The task of structural modeling should be undertaken by an engineer (seismic structural analyst) who is familiar with the basic theory of structural dynamics as well as the finite element structural analysis. The structural analyst should work closely with the principal design engineer in order to develop an understanding of the basic functions and the dynamic interactions among the various components of the structure. In particular, interaction effects of the foundation supporting the hydraulic structure and of the impounded, surrounding, or contained water should be accounted for. However, the structural model selected should be consistent with the level of refinement used in specifying the earthquake ground motion, and should always start with the simplest model possible. Classifications, unit weights, and dynamic modulus and damping properties of the backfill soils and the soil or rock foundation are provided by the geotechnical engineer or engineering geologist member of the project team. Various aspects of the structural modeling and the way seismic input is applied to the structure are discussed in Chapter 2.

e. Prediction of earthquake response of structure. After constructing the structural models, the seismic structural analyst should perform appropriate analyses to predict the earthquake response of the structure. Prediction of the earthquake response includes the selection of a method of analysis covered in paragraph 1-7, formulation of structural mass and stiffness to obtain vibration properties, specification of damping, definition of earthquake loading and combination with static loads, and the computation of response quantities of interest. The analysis should start with the simplest method available and progress to more refined types as needed. It may begin with a pseudo-static analysis performed by hand or spreadsheet calculations, and end with more refined linear elastic response-spectrum and time-history analyses carried out using appropriate computer programs. The required material parameters are formulated initially based on preliminary values from the available data and past experience, but may need adjustment if the analysis shows strong sensitivity to certain parameters, or new test data become available. Damping values for the linear analysis should be selected consistent with the induced level of strains and the amount of joint opening or cracking and yielding that might be expected. Seismic loads should be combined with the most probable static loads, and should include multiple components of the ground motion when the structure is treated as a two-dimensional (2-D) or three-dimensional (3-D) model. In the modal superposition method of dynamic analysis, the number of vibration modes should be selected according to the guidelines discussed in Chapter 2, and response quantities of interest should be determined based on the types of information needed for the design or the safety evaluation. In simplified procedures, the earthquake loading is represented by the equivalent lateral forces associated with the fundamental mode of vibration, where the resultant forces are computed from the equations of equilibrium.

f. Interpretation and evaluation of results.

(1) Responsibilities. The seismic structural analyst and the principal design engineer are the primary personnel responsible for the interpretation and evaluation of the results. The final evaluation of seismic

performance for damaging earthquakes should include participation by experienced structural earthquake engineers.

(2) Interpretation and evaluation. The interpretation of analysis results should start with the effects of static loads on the structure. The application of static loads and the resulting deflections and stresses (or forces) should be thoroughly examined to validate the initial stress conditions. The earthquake performance of the structure is then evaluated by combining the initial static stresses (or forces) with the dynamic stresses (or forces) due to the earthquake. The evaluation for the linear elastic analysis is carried out by comparing computed stresses for unreinforced concrete (URC) or section forces and deformations for reinforced concrete (RC) with the allowable stress values or the supplied capacities, in accordance with the performance goals set forth in Chapter 2. However, in view of the fact that the predicted earthquake response of the structure is based on numerous assumptions, each of which has a limited range of validity, the evaluation procedure should not be regarded as absolute. The final evaluation therefore should consider the uncertainties associated with the earthquake ground motions, accuracy of the analysis techniques, level of foundation exploration, testing, and confidence in material properties, as well as limitations of the linear analysis and engineering judgment to predict nonlinear behavior.

1-7. Summary of Seismic Analysis of Concrete Hydraulic Structures

a. General. Hydraulic structures traditionally have been designed based on the seismic coefficient method. This simple method is now considered inadequate because it fails to recognize dynamic behavior of the structures during earthquake loading. The seismic coefficient method should be used only in the preliminary design and evaluation of hydraulic structures for which an equivalent static force procedure based on the vibration properties of the structure has not yet been formulated. The final design and evaluation of hydraulic structures governed by seismic loading should include response spectra and, if needed, acceleration time-histories as the seismic input and response spectrum or time-history method of analysis for predicting the dynamic response of the structure to this input. With recent advances in the estimation of site-specific ground motions and in structural dynamic computer analysis techniques, the ability to perform satisfactory and realistic analyses has increased. This manual presents improved guidelines for the estimation of site-specific ground motions and the prediction of dynamic response for the design and seismic safety evaluation of hydraulic structures.

b. Types of hydraulic structures. The general guidelines provided in this manual apply to concrete hydraulic structures including locks, intake towers, earth retaining structures, arch dams, conventional and Roller Compacted Concrete (RCC) gravity dams, powerhouses, and critical appurtenant structures.

c. Design criteria. The design and evaluation of hydraulic structures for earthquake loading must be based on appropriate criteria that reflect both the desired level of safety and the nature of the design and evaluation procedures (ER 1110-2-1806). The first requirement is to establish earthquake ground motions to be used as the seismic input by considering safety, economics, and the designated operational functions. The second involves evaluating the earthquake performance of the structure to this input by performing a linear elastic dynamic analysis based on a realistic idealization of the structure, foundation, and water.

d. Design earthquakes. The design earthquakes for hydraulic structures are the OBE and the MDE. The actual levels of ground motions for these earthquakes depend on the type of hydraulic structure under consideration, and are specified in the seismic design guidance provided for a particular structure in conjunction with ER 1110-2-1806.

(1) Operating basis earthquake (OBE). The OBE is an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50 percent probability of exceedance during the

service life. The associated performance requirement is that the project function with little or no damage, and without interruption of function.

(2) Maximum design earthquake (MDE). The MDE is the maximum level of ground motion for which the structure is designed or evaluated. The associated performance requirement is that the project performs without catastrophic failure, such as uncontrolled release of a reservoir, although severe damage or economic loss may be tolerated. The MDE is set equal to the maximum credible earthquake (MCE) or to a lesser earthquake, depending on the critical nature of the structure (see ER 1110-2-1806 and paragraph 2-4b).

(3) The MCE is defined as the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence.

e. Earthquake ground motion(s). The ground motions for the design earthquakes are defined in terms of smoothed elastic response spectra and, if required, also in terms of acceleration time-histories. Standard ground motions selected from published ground motion maps can be used in preliminary and screening studies, and for final design or evaluation in areas of low to moderate seismicity where the earthquake loading does not control the design. Site-specific ground motions, as described in Chapter 3, are required for projects with high to significant hazard potential in case of failure and located in areas of high seismicity, and in areas of moderate seismicity where the earthquake loading controls the design (ER 1110-2-1806).

f. Structural idealization. The structural idealization should start with the simplest model possible and, if required, progress to a 2-D or a more comprehensive 3-D model. The structural model should represent the important features of the dynamic behavior of the structure including its interaction with the foundation and the water. It should also be consistent with the design and evaluation objectives, that is, to reflect the relative accuracy suitable for the type of seismic input used as well as the type of studies performed, i.e., feasibility, preliminary, or final study. For example, one-dimensional (1-D) models are used for the preliminary design and evaluation, whereas depending on geometry of the structure, 2-D or 3-D models are used in the final phase of the study.

(1) Simplified models. Simplified models are based on the equivalent lateral force procedures, where the earthquake response of the structure is obtained directly from the response spectra. In most cases only the fundamental mode of vibration, but sometimes the second mode as well, is considered. However, only the fundamental mode is adjusted to account for the effects of structure-foundation and structure-water interaction.

(2) Two-dimensional models. 2-D models including the structure-foundation and structure-water interaction effects are developed using the finite element (FE) procedures. They are employed in the final or preliminary study of structures for which simplified models have not yet been formulated. The seismic input consists of response spectra (or acceleration time-histories) for the vertical and one horizontal components of ground motion. The seismic input is applied either at the base of the composite structure-foundation model or at the base of the structure if the substructure method of analysis is used.

(3) Three-dimensional models. Hydraulic structures with complicated 3-D geometry should be idealized as 3-D models and analyzed for all three components of the earthquake ground motion. The model should be developed using FE procedures and account for the effects of structure-foundation and structure-water interaction. The seismic input in the form of response spectra or acceleration time-histories is applied along three principal axes of the structure either at the base of the composite structure-foundation model or at the base of the structure if the foundation region is analyzed separately.

g. Dynamic analysis procedures. Linear dynamic analysis procedures are presently used for earthquake-resistant design and safety evaluation of hydraulic structures. The linear dynamic analysis is performed using the response spectrum or the time-history modal superposition method. The primary feature of the modal analysis is that the total response of a structure is obtained by combining the response of its individual modes of vibration, calculated separately. The response spectrum analysis is adequate for structures whose responses to earthquakes are within the linear elastic range. But for structures for which the cracking strength of the concrete and yield strength of the reinforcing steels may be exceeded under a major earthquake, a linear time-history analysis provides additional information that is essential to approximating the damage or expected level of inelastic response behavior.

(1) Response spectrum analysis. In the response spectrum analysis, the maximum response of the structure to earthquake excitation is evaluated by combining the maximum responses from individual modes and multicomponent input. All response quantities computed in this manner are positive and require careful examination and interpretation. The accuracy of the results depends on the number of vibration modes considered and the methods of combination used for the modal and multicomponent earthquake responses.

(2) Time-history analysis. Linear time-history analysis involves computation of the complete response history of the structure to earthquakes, and not just the maximum values. The results of such analysis serve to demonstrate the general behavior of the seismic response, and combined with rational interpretation and judgment can provide a reasonable estimate of the expected inelastic behavior or damage, when the cracking or other form of nonlinearity is considered to be slight to moderate. Prediction of the actual damage that could occur during major earthquakes can only be estimated using more complicated nonlinear analyses, but approximate assessment can still be made using the analysis discussed in paragraph 2-4b(3)(a) and (b). The complete nonlinear analysis of hydraulic structures is not currently practical; only limited aspects of the nonlinear response behavior such as joint opening and sliding of blocks can be considered.

h. Interpretation and evaluation of results. The evaluation of earthquake performance of hydraulic structures is currently based on the numerical results of linear dynamic analyses, in which the calculated stresses for URC or section forces and deformations for RC are compared with the allowable stress values or the supplied moment and shear capacities. New hydraulic structures should resist the OBE excitation within the elastic range of the element stresses (or section forces) to avoid structural damage or yielding. However, existing hydraulic structures in high seismic hazard regions may be allowed to respond to the OBE excitation within the *nearly elastic* range; that is, minor local damage or yielding is permitted if retrofit to preclude damage is deemed uneconomical. The evaluation for the severe MDE excitation is more complicated because the dynamic response is expected to exceed the linear elastic limits, resulting in damage and inelastic behavior. In such cases, the extent of damage for URC hydraulic structures is normally estimated based on the results of linear response history analysis together with engineering judgment and other considerations discussed in paragraph 2-8a(4). For RC hydraulic structures undergoing inelastic deformations, approximate postelastic dynamic analyses are performed to ensure that the inelastic demands of the severe MDE excitation can be resisted by the available capacity of the structure. The postelastic analysis discussed in paragraph 2-4b(3)(b) is a step-by-step linear analysis with revised stiffness or resistance characteristics of all structural members that have reached their yielding capacities. The stiffness modification and analysis of the modified structure are repeated until no further yielding will occur or the structure reaches a limit state with excessive distortions, mechanism, or instability.

1-8. Summary of Development of Site-Specific Response Spectra for Seismic Analysis of Structures

a. Factors affecting earthquake ground motion. It has been well recognized that earthquake ground motions are affected by earthquake source conditions, source-to-site transmission path properties, and site conditions. The source conditions include stress conditions, source depth, size of rupture area, amount of rupture displacement, rise time, style of faulting, and rupture directivity. The transmission path properties include the crustal structure and the shear wave velocity and damping characteristics of the crustal rock. The site conditions include the rock properties beneath the site to depths of up to 2 km, the local soil conditions at the site for depths of up to several hundred feet, and the topography of the site. In current ground motion estimation relationships, the effects of source, path, and site are commonly represented in a simplified manner by earthquake magnitude, source-to-site distance, and local subsurface conditions. Due to regional differences in some of the factors affecting earthquake ground motions, different ground motion attenuation relationships have been developed for western United States (WUS) shallow crustal earthquakes, eastern United States (EUS) earthquakes, and subduction zone earthquakes (which, in the United States, can occur in portions of Alaska, northwest California, Oregon, and Washington). It is also recognized that ground motions in the near-source region of earthquakes may have certain characteristics not found in ground motions at more distant sites, especially a high-energy intermediate-to-long-period pulse that occurs when fault rupture propagates toward a site.

b. Basic approaches for developing site-specific response spectra. There are two basic approaches to developing site-specific response spectra: deterministic and probabilistic. In the deterministic approach, termed deterministic seismic hazard analysis, or DSHA, typically one or more earthquakes are specified by magnitude and location with respect to a site. Usually, the earthquake is taken as the MCE and assumed to occur on the portion of the source closest to the site. The site ground motions are then estimated deterministically, given the magnitude and source-to-site distance. In the probabilistic approach, termed probabilistic seismic hazard analysis, or PSHA, site ground motions are estimated for selected values of the probability of ground motion exceedance in a design time period or for selected values of return period of ground motion exceedance. A PSHA incorporates the frequency of occurrence of earthquakes of different magnitudes on the various seismic sources, the uncertainty of the earthquake locations on the sources, and the ground motion attenuation including its uncertainty. Guidance for using both of these approaches is presented in Chapter 3 and is briefly summarized below.

(1) Deterministic approach for developing site-specific response spectra. Deterministic estimates of response spectra can be obtained by either Approach 1, anchoring a response spectral shape to the estimated peak ground acceleration (PGA); or Approach 2, estimating the response spectrum directly. When Approach 1 is followed, it is important to consider the effects of various factors on spectral shape (e.g., regional tectonic environment, earthquake magnitude, distance, local soil or rock conditions). Because of the significant influence these factors have on spectral shape and because procedures, data, and relationships are now available to estimate response spectra directly, Approach 2 should be used. Approach 1 can be used for comparison. The implementation of Approach 2 involves the following:

(a) Using response spectral attenuation relationships of ground motions (attenuation relationships are now available for directly estimating response spectral values at specific periods of vibration).

(b) Performing statistical analyses of response spectra of ground motion records.

(c) Applying theoretical (numerical) ground motion modeling.

When soil deposits are present at a site and response spectra on top of the soil column are required (rather than or in addition to spectra on rock), then either empirically based approaches and/or analytical procedures can be used to assess the local soil amplification effects. Empirically based approaches rely on recorded ground motion data and resulting empirical relationships for similar soil conditions. Analytical procedures involve modeling the dynamic properties of the soils and using dynamic site response analysis techniques to propagate motions through the soils from the underlying rock.

(2) Probabilistic approach for developing site-specific response spectra. Similar to a deterministic analysis, a probabilistic development of a site-specific response spectrum can be made by either Approach 1, anchoring a spectral shape to a PGA value, or Approach 2, developing the spectrum directly. In Approach 1, PSHA is carried out for PGA, and an appropriate spectral shape must then be selected. The selection of the appropriate shape involves the analysis of earthquake sizes and distances contributing to the seismic hazard. In Approach 2, the PSHA is carried out using response spectral attenuation relationships for each of several periods of vibration. Drawing a curve connecting the response spectral values for the same probability of exceedance gives a response spectrum having an equal probability of exceedance at each period of vibration. The resulting spectrum is usually termed an equal-hazard spectrum. Approach 2 should be used because response spectral attenuation relationships are now available and the use of these relationships directly incorporates into the analysis the influence of different earthquake magnitudes and distances on the results for each period of vibration.

c. *Developing acceleration time-histories*. When acceleration time-histories are required for the structure dynamic analysis, they should be developed to be consistent with the design site-specific response spectrum. They should also have an appropriate duration of shaking (duration of shaking is strongly dependent on earthquake magnitude). The two general approaches to developing acceleration time-histories are selecting a suite of recorded motions that, in aggregate, have spectra that envelope the design spectrum; or synthetically modifying one or more recorded motions to produce motions having spectra that are a close match to the design spectrum ("spectrum matching" approach). For either approach, when near-source ground motions are modeled, it is desirable to include a strong intermediate-to-long-period pulse to model this characteristic that is observed in near-source ground motions.

1-9. Terminology

Appendix I contains defiinitions of terms that relate to Response Spectra and Seismic Analysis for Hydraulic Structures.

Chapter 2
Seismic Analysis of Concrete Hydraulic Structures

2-1. Introduction

a. General. This chapter provides structural guidance for the use of response spectra for the seismic design and evaluation of the Corps of Engineers hydraulic structures. These include locks, intake towers, earth retaining structures, arch dams, conventional and RCC gravity dams, powerhouses, and critical appurtenant structures. The specific requirements are provided for the structures built on rock, such as the arch and most gravity dams, as well as for those built on soil or pile foundations, as in the case of some lock structures. The response spectrum method of seismic design and evaluation provisions for building-type structures are summarized in paragraph 2-10.

b. Interdisciplinary collaboration. A complete development and use of response spectra for seismic design and evaluation of hydraulic structures require the close collaboration of a project team consisting of several disciplines.

(1) Project team. The specialists in the disciplines of seismology, geophysics, geology, and geotechnical engineering develop design earthquakes and the associated ground motions, with the results presented and finalized in close cooperation with structural engineers. The materials engineer and geotechnical specialists specify the material properties of the structure and of the soils and rock foundation. The structural engineer in turn has the special role of explaining the anticipated performance and the design rationale employed to resist the demands imposed on the structure by the earthquake ground motions.

(2) Ground motion studies. As discussed in Chapter 3, the seismic input in the form of site-specific response spectra is developed using a deterministic or a probabilistic approach. Both methods require the following three main items to be clearly addressed and understood so the project team members have a common understanding of the design earthquakes: seismic sources, i.e., faults or source areas that may generate earthquakes; maximum earthquake sizes that can occur on the identified sources and their frequency of occurrence; and attenuation relationships for estimation of ground motions in terms of magnitude, distance, and site conditions. The results of ground motion studies should be presented as required in ER 1110-2-1806. For a DSHA mean and 84th percentile, response spectra for the MCE should be presented. For a PSHA, response spectra should be presented as equal hazard spectra at various levels of probability and damping, as described in ER 1110-2-1806 and Chapter 3. Acceleration time-histories based on natural or synthetic accelerograms may also be required. The assumptions and methodology used to perform a DSHA and PSHA should be explained, and the uncertainties associated with the selection of input parameters should be presented in the report.

2-2. General Concepts

Two essential problems must be considered in the seismic analysis and design of structures: definition of the expected earthquake input motion and the prediction of the response of the structure to this input. The solutions to these problems are particularly more involved for the structures founded on soil or pile foundations and for those built on rock sites with complicated topography as in the case of arch dams.

a. Input motion(s). A general description of the factors affecting the earthquake input motions to be used in the design and evaluation of structures is demonstrated in Figure 2-1. The base rock motion

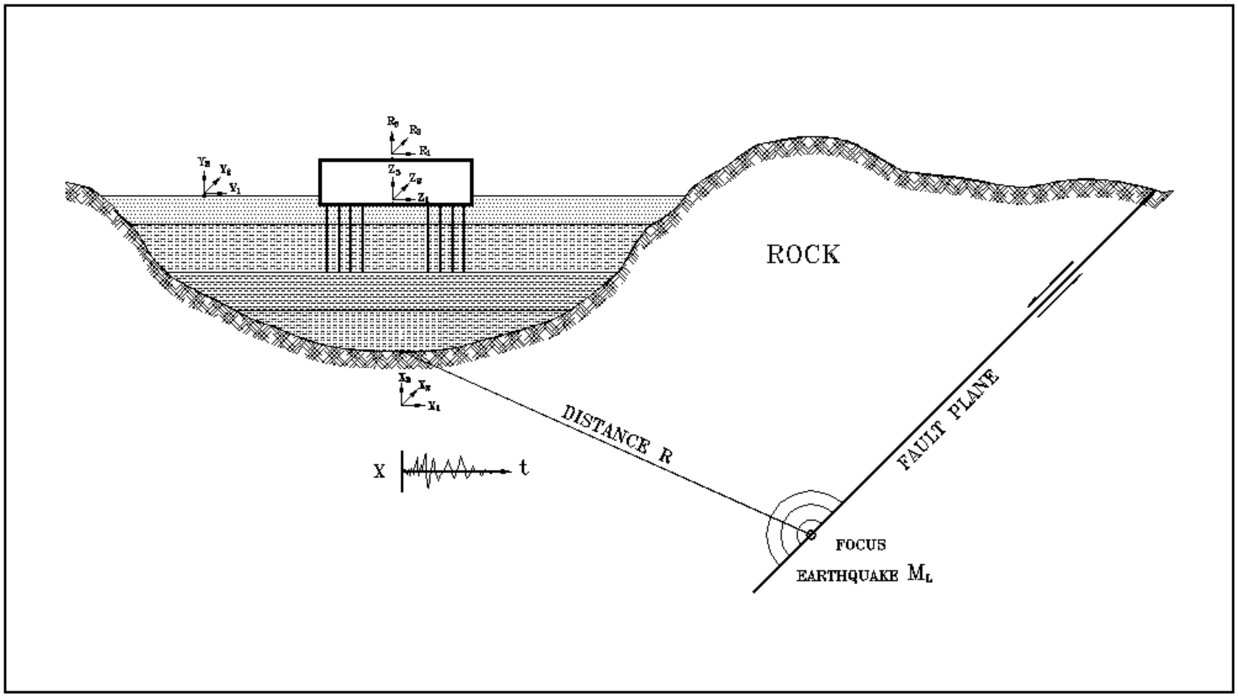


Figure 2-1. Factors affecting seismic input motion for a structure founded on soil-pile foundation

$X_i(i = 1,2,3)$ is estimated from the study of regional geologic setting, historic seismicity of the area, and the geologic structure along the path from source to site. The characteristics of this motion, however, are affected by the local soil conditions as it travels to the free ground surface. Thus, the resulting free-field motion Y_i ($i = 1,2,3$), in the absence of the structure, differs from X_i in terms of the peak amplitude, the frequency content, and the spatial distribution of the motion characteristics. In addition, the dynamic interaction of the structure with the soil foundation produces a further change of the seismic motions, leading to Z_i ($i = 1,2,3$) at the soil-structure interface. Depending on the method of analysis adopted, one of these motions is selected as the earthquake input in the actual dynamic analysis of the structure. If X_i or Y_i is selected, the soil foundation is modeled as part of the structure, and a direct method of soil-structure interaction (SSI) analysis is performed. Alternatively, the structure and the soil region may be treated as two separate substructures. First the soil region is analyzed with the mass of the structure set to zero, to obtain ground motion Z_i at the soil-structure interface (kinematic interaction). The same model is also used to determine the dynamic stiffness of the soil region. Then Z_i is used as the input motions in the subsequent earthquake response analysis of the structure whose stiffness is now being combined with the dynamic stiffness of the soil region, and its mass being considered. To estimate these ground motions, however, many aspects of the problem such as the seismic environment, dynamic soil properties, site response, and the structural analysis must be considered. The solution thus requires close cooperation among the geologist, seismologist, and geotechnical and structural engineers to achieve satisfactory results.

b. Structural response. The second problem involves prediction of the response of the structure to the specified input motion. This requires development of a structural model, specification of material properties and damping, and calculation of the response, taking into account the dynamic interactions with the foundation, the water, and the backfill soils. Depending on complexity of the structure and intensity of the earthquake, a simple or more advanced modeling and analysis may be required. In either case the analysis should consist of the following steps, except that the level of effort may be different for simple and more refined analyses:

(1) Establishment of earthquake design criteria.

(2) Development of design earthquakes and associated ground motions.

(3) Establishment of analysis procedure.

(4) Development of structural models.

(5) Prediction of earthquake response of the structure.

(6) Interpretation and evaluation of results.

2-3. Design Criteria

The design and evaluation of hydraulic structures for earthquake loading must be based on appropriate criteria that reflect both the desired level of safety and the choice of the design and evaluation procedures (ER 1110-2-1806). The first requirement is to establish design earthquake ground motions to be used as the seismic input by giving due consideration to the consequences of failure and the designated operational function. Then the response of the structure to this seismic input must be calculated taking into account the significant interactions with the rock, soil, or pile foundation as well as with the impounded, or surrounding and contained water. The analysis should be formulated using a realistic idealization of the structure-water-foundation system, and the results are evaluated in view of the limitations, assumptions, and uncertainties associated with the seismic input and the method of analysis.

2-4. Design Earthquakes

a. Operating basis earthquake (OBE).

(1) Definition and performance. The OBE is an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50 percent probability of exceedance during the service life. (This corresponds to a return period of 144 years for a project with a service life of 100 years.) The associated performance requirement is that the project function with little or no damage, and without interruption of function. The purpose of the OBE is to protect against economic losses from damage or loss of service. Therefore alternative choices of return period for the OBE may be based on economic considerations. In a site-specific study the OBE is determined by a PSHA (ER 1110-2-1806).

(2) Analysis. For the OBE, the linear elastic analysis is adequate for computing seismic response of the structure, and the simple stress checks in which the predicted elastic stresses are compared with the expected concrete strength should suffice for the performance evaluation. Structures located in regions of high seismicity should essentially respond elastically to the OBE event with no disruption to service, but limited localized damage is permissible and should be repairable. In such cases, a low to moderate level of damage can be expected, but the results of a linear time-history analysis with engineering judgment may still be used to provide a reasonable estimate of the expected damage.

b. Maximum design earthquake (MDE).

(1) Definition and performance. The MDE is the maximum level of ground motion for which a structure is designed or evaluated. The associated performance requirement is that the project performs without catastrophic failure, such as uncontrolled release of a reservoir, although severe damage or economic loss may be tolerated. The MDE can be characterized as a deterministic or probabilistic event (ER 1110-2-1806).

(a) For critical structures the MDE is set equal to the MCE. Critical structures are defined in ER 1110-2-1806 as structures of high downstream hazard whose failure during or immediately following an earthquake could result in loss of life. The MCE is defined as the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence (ER 1110-2-1806).

(b) For other than critical structures the MDE is selected as a lesser earthquake than the MCE, which provides for an economical design meeting specified safety standards. This lesser earthquake is chosen based upon an appropriate probability of exceedance of ground motions during the design life of the structure (also characterized as a return period for ground motion exceedance).

(2) Nonlinear response. The damage during an MDE event could be substantial, but it should not be catastrophic in terms of loss of life, economics, and social and environmental impacts. It is evident that a realistic design criterion for evaluation of the response to damaging MDEs should include nonlinear analysis, which can predict the nature and the extent of damage. However, a complete and reliable nonlinear analysis that includes tensile cracking of concrete, yielding of reinforcements, opening of joints, and foundation failure is not currently practical. Only limited aspects of the nonlinear earthquake response behavior of the mass concrete structures such as contraction joint opening in arch dams, tensile cracking in concrete gravity dams, and sliding of concrete monoliths have been investigated previously. There is a considerable lack of knowledge with respect to nonlinear response behavior of the hydraulic structures. Any consideration of performing nonlinear analysis for hydraulic structures should be done in consultation with CECW-ED.

(3) Performance evaluation. The earthquake performance evaluation of the response of hydraulic structures to a damaging MDE is presently based on the results of linear elastic analysis. In many cases, a linear elastic analysis can provide a reasonable estimate of the level of expected damage when the cracking, yielding, or other forms of nonlinearity are considered to be slight to moderate.

(a) URC. For URC hydraulic structures subjected to a severe MDE, the evaluation of damage using the linear time-history analysis may still continue. The evaluation, however, must be based on a rational interpretation of the results by giving due consideration to several factors including number and duration of stress excursions beyond the allowable limits, the ratio of computed to allowable values, simultaneous stress distributions at critical time-steps, size and location of overstressed area, and engineering judgment.

(b) RC. Such evaluation for the RC hydraulic structures should include approximate postelastic analysis of the system considering ductility and energy dissipation beyond yield. First the section forces for critical members are computed using the linear elastic analysis procedure described in this manual. These forces are defined as the force *demands* imposed on the structure by the earthquake. Next the yield or plastic capacities at the same locations are computed and defined as the force *capacities*. Finally, the ratio of force demands to force capacities is computed to establish the *demand-capacity ratios* for all the selected locations. The resulting demand-capacity ratios provide an indication of the ductility that may be required for the structural members to withstand the MDE level of ground motion. If the computed demand-capacity ratios for a particular structure exceed the limits set forth in the respective design documents for that structure, approximate postelastic analyses should be performed to ensure that the inelastic demands of the MDE excitation on the structure can be resisted by the supplied capacity. This evaluation consists of several equivalent linear analyses with revised stiffness or resistance characteristics of all structural members that have reached their yielding capacities. The stiffness modification and analysis of the modified structure are repeated until no further yielding will occur or the structure reaches a limit state with excessive distortions, mechanism, or instability.

2-5. Earthquake Ground Motions

Earthquake ground motions for analysis of hydraulic structures are usually characterized by peak ground acceleration, response spectra, and acceleration time-histories. The peak ground acceleration (usually as a fraction of the peak) is the earthquake ground motion parameter usually used in the seismic coefficient method of analysis. The earthquake ground motions for dynamic analysis, as a minimum, should be specified in terms of response spectra (Figure 2-2). A time-history earthquake response analysis, if required, should be performed using the acceleration time-histories. The standard response spectra are described in the following paragraphs, and procedures for estimating site-specific response spectra are discussed in Chapter 3.

a. Elastic design response spectra. Elastic design response spectra of ground motions can be defined by using standard or site-specific procedures. As illustrated in Figure 2-2, elastic design response spectra represent maximum responses of a series of single-degree-of-freedom (SDOF) systems to a given ground motion excitation (Ebeling 1992; Chopra 1981; Clough and Penzien 1993; Newmark and Rosenblueth 1971). The maximum displacements, maximum pseudo-velocities, and maximum pseudo-accelerations presented on a logarithmic tripartite graph provide advance insight into the dynamic behavior of a structure. For example, Figure 2-2 shows that at low periods of vibration (<0.05 sec), the spectral or pseudo-accelerations approach the PGA, an indication that the rigid or very short period structures undergo the same accelerations as the ground. This figure also shows that structures with periods in the range of about 0.06 to 0.5 sec are subjected to amplified accelerations and thus higher earthquake forces, whereas the earthquake forces for flexible structures with periods in the range of about 1 to 20 sec are reduced substantially but their maximum displacements exceed that of the ground. In the extreme, when the period exceeds 20 sec, the structure experiences the same maximum displacement as the ground. The response spectrum amplifications depend on the values of damping and are significantly influenced by the earthquake magnitude, source-to-site distance, and the site conditions (Chapter 3).

(1) Standard or normalized response spectra. The standard response spectra described in this section are to be used in accord with ER 1110-2-1806 and follow-up guidance. The development starts with the spectral acceleration ordinates obtained from the National Earthquake Hazards Reduction Program (NEHRP) hazard maps. The site coefficients to be used with the hazard maps to develop standard response spectra for various soil profiles, as well as the methodology to construct response spectra at return periods other than those given in NEHRP, are provided in the guidance in ER 1110-2-1806.

(2) Site-specific response spectra.

(a) Site-specific procedures to produce design response spectra are to be used in accord with ER 1110-2-1806. Site-specific response spectra correspond to those expected on the basis of the seismological and geological calculations for the site. The procedures described in Chapter 3 use either the deterministic or probabilistic method to develop site-specific spectra.

(b) While the deterministic method provides a single estimate of the peak ground acceleration and response spectral amplitudes, the probabilistic method estimates these parameters as a function of probability of exceedance or return period. To select the return period to use for the OBE and MDE, see the definitions of these design earthquakes in paragraphs 2-4a and 2-4b, respectively. The resulting response spectra for the selected return period should then be used as input for quantifying the seismic loads required for the design and analysis of structures.

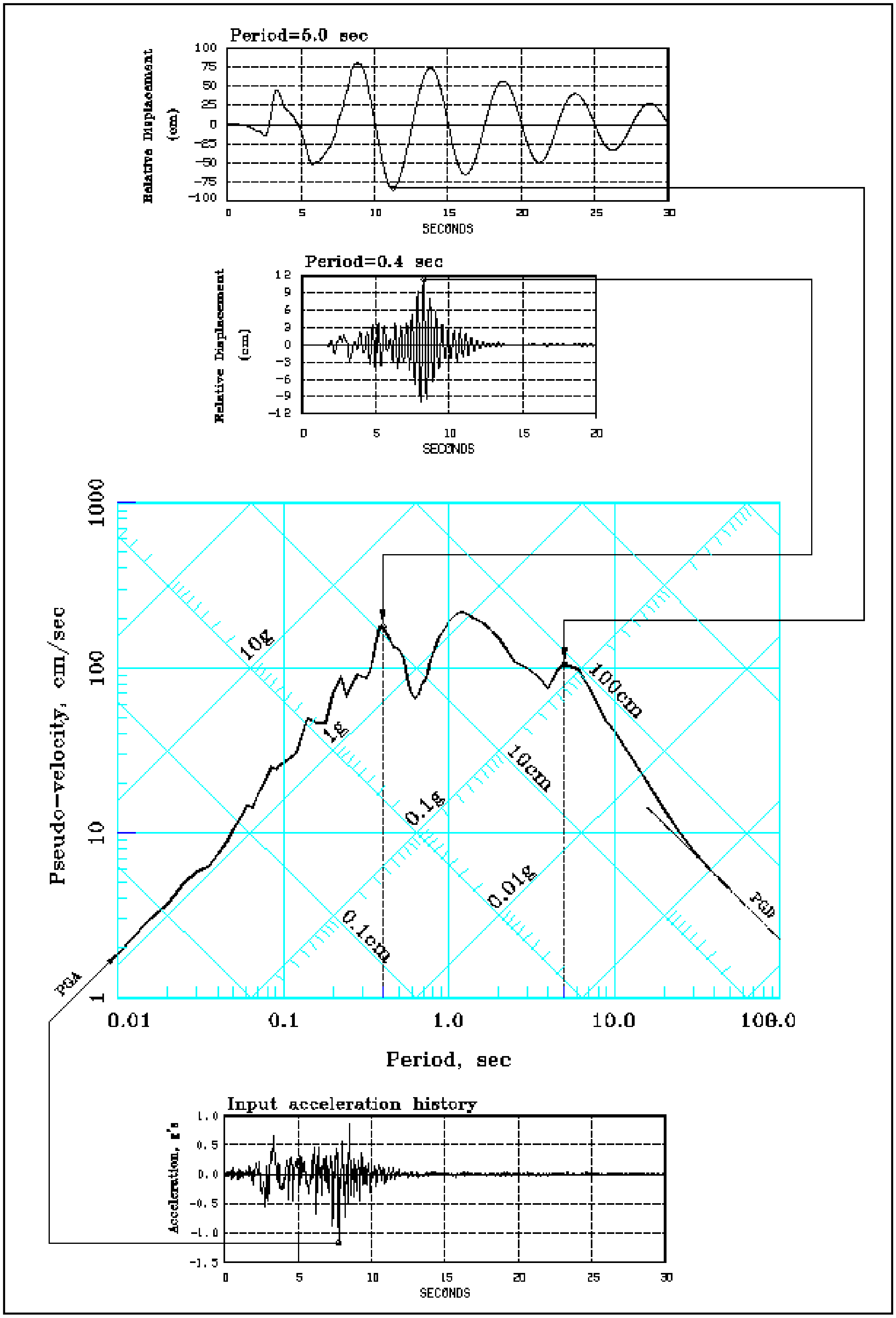


Figure 2-2. Construction of tripartite elastic design response spectrum

b. Acceleration time-histories. Various procedures for developing representative acceleration time-histories at a site are described in Chapter 3. Whenever possible, the acceleration time-histories should be selected to be similar to the design earthquake in the following aspects: tectonic environment, earthquake magnitude, fault rupture mechanism (fault type), site conditions, design response spectra, and duration of strong shaking. Since it is not always possible to find records that satisfy all of these criteria, it is often necessary to modify existing records or develop synthetic records that meet most of these requirements.

2-6. Establishment of Analysis Procedures

Seismic analysis of hydraulic structures should conform to the overall objectives of new designs and satisfy the specific requirements of safety evaluation of existing structures. The choice of analysis procedures may influence the scope and nature of the seismic input characterization, design procedures, specification of material properties, and evaluation and interpretation of the results. Simple procedures require fewer and easily available parameters, while refined analyses usually need more comprehensive definition of the seismic input, structural idealization, and material properties. The analysis should begin with the simplest procedures possible and then, if necessary, progress to more refined and advanced types. Simplified procedures are usually adequate for the feasibility and preliminary studies, whereas refined procedures are more appropriate for the final design and safety evaluation of structures. The simplified analysis also serves to assess the need for a more elaborate analysis and provide a baseline for comparison with the results obtained from the more elaborate analyses.

2-7. Structural Idealization

Structural models should be developed by giving careful consideration to the geometry, stiffness, and mass distributions, all of which affect the dynamic characteristics of the structure. The engineering judgment and knowledge of the dynamics of structures are required to develop a satisfactory model that is both simple and representative of the most important dynamic behavior of the structure. Depending on its level of complexity, a hydraulic structure may be represented by a simplified one-dimensional model, a planar or 2-D model whose deformations are restricted in a plane, or by a more elaborate 3-D model to account for its 3-D behavior.

a. Simplified models. Structures with regular geometry and mass distribution along one axis may be idealized by simplified models using the beam theory. The simplified model should approximately represent the significant features of the dynamic response of the structure including the fundamental period and mode shape, as well as the effects of structure-foundation and structure-water interaction. Two such simplified models have been developed for the free-standing intake towers and the nonoverflow gravity dam sections. In both cases, the simplified models were formulated based on the results of finite element analyses that rigorously accounted for the structure-water-foundation interaction effects, as well as for the reservoir bottom energy absorption for the gravity dams.

(1) Simplified model for intake towers. The preliminary design and safety evaluation of the free-standing and regular intake towers may be conducted using the simplified model shown in Figure 2-3. A step-by-step analysis procedure for this cantilever beam model is provided in Goyal and Chopra (1989) and Appendix H. Some important features and assumptions of this approximate model are as follows:

(a) It is applicable to towers with regular geometry in plan and elevation.

(b) Only flexural deformations are considered.

(c) Seismic response is calculated for the first two modes of vibration.

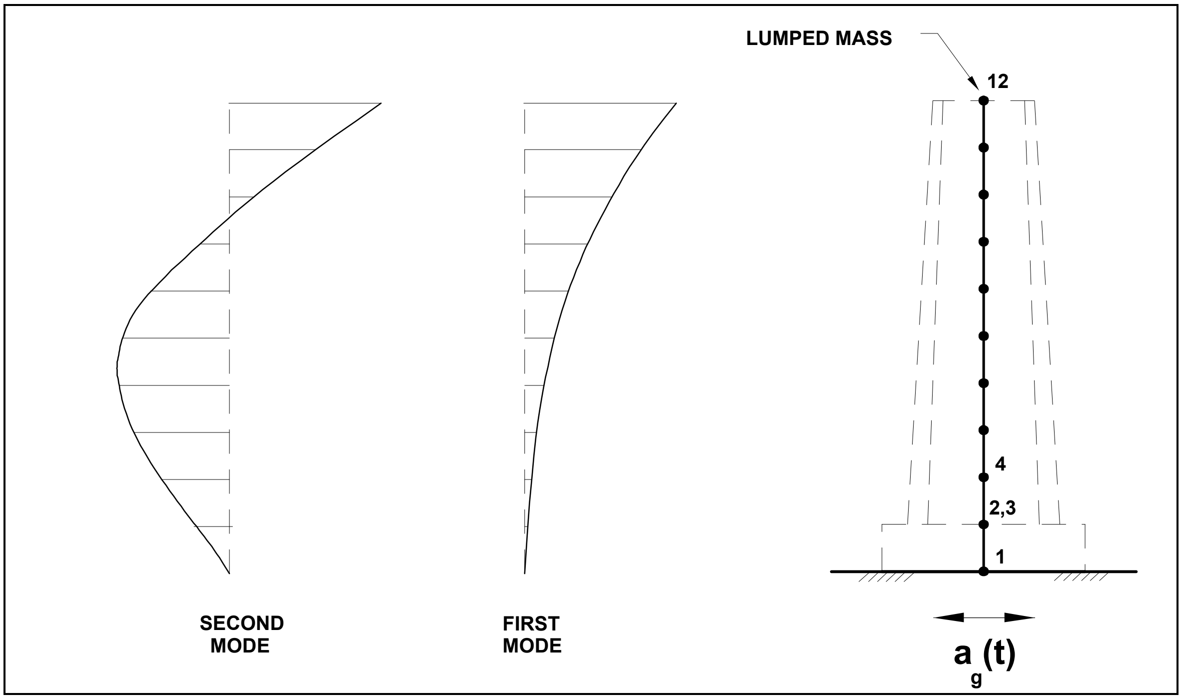


Figure 2-3. Simplified model of intake tower ($a_g(t)$ = ground motion acceleration)

(d) Foundation-structure interaction effects are considered only for the first mode of vibration.

(e) Interaction between the tower and the inside and outside water is represented by the added mass assumption.

(f) The effects of vertical component of ground motion are ignored.

Note that slender towers with cross-section dimensions 10 times less than the height of the structure can usually be adequately represented solely by the flexural deformations of the tower. However, the effects of shear deformations on vibration frequencies and section forces, especially for higher modes, are significant when the cross-section dimensions exceed 1/10 of the tower height and should be included in the analysis. The effects of shear deformation can be incorporated in the analysis if a computer program with beam elements including shear deformation is used. The earthquake response for this simplified model should be calculated for the combined effects of the two horizontal components of the ground motions. The maximum shear forces, moments, and stresses for each lateral direction are computed separately using the specified response spectrum and the calculated vibration properties associated with that direction. The total response values of the tower are then obtained by combining the responses caused by each of the two components of the earthquake ground motion, as discussed in paragraph 2.8a(2)(f).

(2) Simplified model for gravity dams. The preliminary design and safety evaluation of gravity dams may start with a simplified model developed by Fenves and Chopra (1986), as shown in Figure 2-4. In this procedure, deformations of the dam monolith are restricted to the fundamental mode of vibration of the dam on rigid foundation rock. Standard values are provided for the fundamental vibration period and mode shape of typical nonoverflow gravity sections. But they are not available for the nonstandard or spillway sections whose geometries substantially differ from that of a typical nonoverflow section. In such cases, the fundamental vibration period and mode shape for the nonstandard section should be

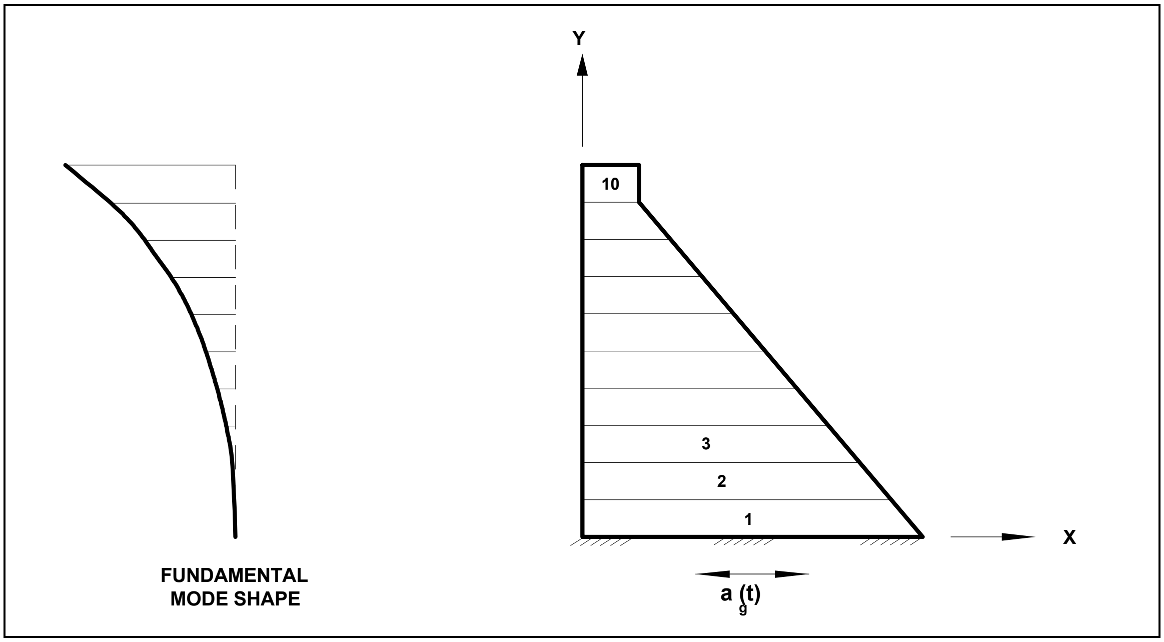


Figure 2-4. Simplified model of gravity dam monolith (Fenves and Chopra (1986), courtesy of Earthquake Engineering Research Center, University of California at Berkeley)

estimated using other procedures before this simplified method could be applied. The most important features of the simplified model are summarized as follows.

(a) Only fundamental mode of vibration is considered; contributions due to higher modes are accounted for by static correction.

(b) The upstream face of the dam is assumed to be vertical or nearly vertical.

(c) The effects of vertical ground motion are ignored.

(d) The interactions with the foundation rock and water are accounted for by adjustment of the vibration period and damping of the fundamental mode. The inertial effects of the added mass of water are considered in terms of additional lateral hydrodynamic forces.

b. Two-dimensional models. 2-D idealization is used to model planar or very long structures. Most Corps of Engineers hydraulic structures are of the latter type such as the retaining walls, gravity dams, outlet tunnels, and lock structures. These structures are usually made of independent segments separated by construction joints, and the loads perpendicular to the long axis are assumed not to vary along each segment. Under these conditions, the structure may be modeled as a 2-D slice using either the plane stress or plane strain elements, as shown in Figure 2-5. The choice of plane stress or plane strain elements depends on whether the stress or strain in the out-of-plane direction can be neglected. In either case, plane strain models should be used to idealize the foundation supporting the structure. 2-D models should be analyzed for two components of the earthquake ground motion applied in the vertical and one horizontal direction. However, the way the seismic input is applied to the structure depends on the type of foundation model being used. Three commonly used foundation models and their associated seismic input for the analysis of typical hydraulic structures are discussed in the following paragraphs.

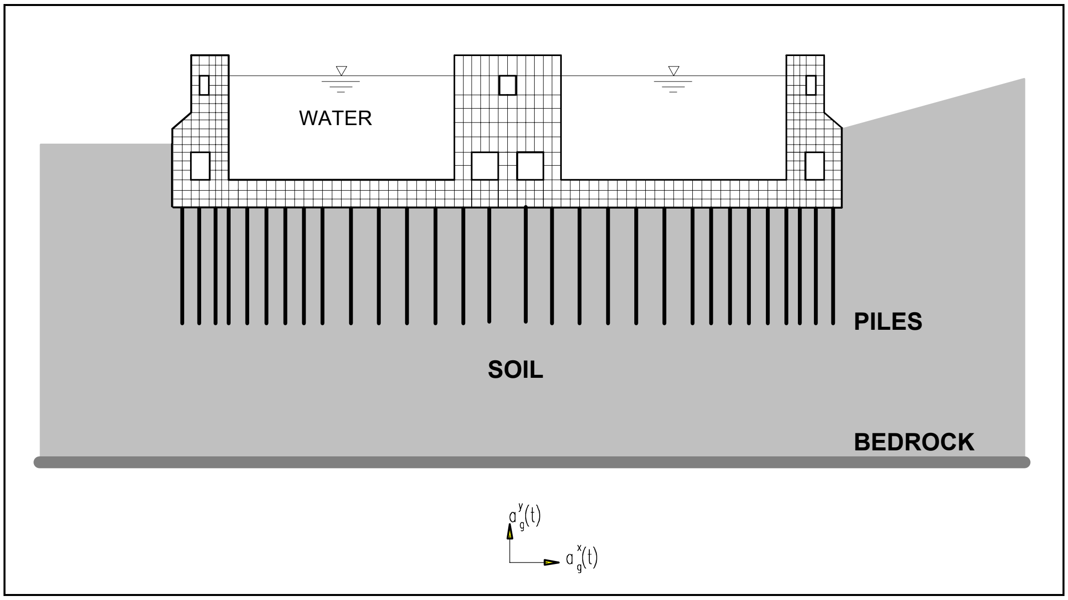


Figure 2-5. 2-D model of W-frame lock

(1) Rigid rock-base excitation. The standard approach to accounting for the effects of the foundation interaction is to analyze the combined structure-foundation system by including an appropriate region of the rock in the finite element idealization, as shown in Figure 2-6a. In this approach, the earthquake motion is represented as a rigid body translation a_R of the basement rock, and either the response spectra or acceleration time-histories are used as input to the model. The characteristics of the specified earthquake ground motion should be similar to the motions recorded on the rock sites. The location of the rigid boundary at the base of the model should be selected consistent with the size and type of the structure being analyzed. The mass of foundation rock should be ignored so that the free-field motions recorded at ground surface are directly applied to the structure without changes, and the spurious reflection effects caused by the rigid boundary assumption are eliminated.

(2) Free-field earthquake excitation. For rock and firm soil sites where similar foundation materials extend to large depths, the foundation region may be idealized as a homogeneous, isotropic, viscoelastic

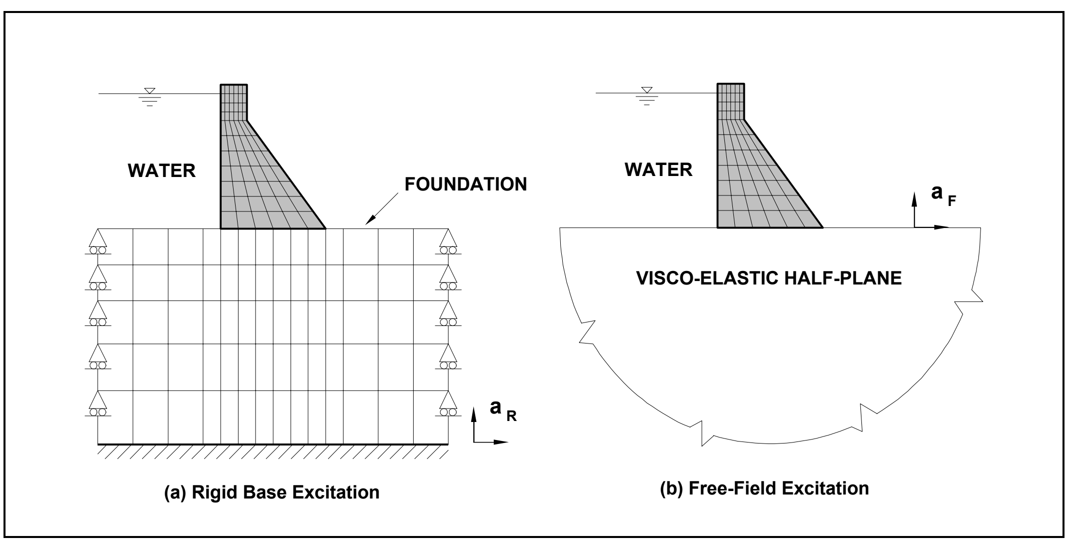


Figure 2-6. Earthquake excitation for rock or firm soil sites

half-plane (Dasgupta and Chopra 1979), as shown in Figure 2-6b. In this case, the structure is supported on the horizontal surface of the foundation, and the earthquake response is formulated with respect to the free-field definition of the ground motion a_F rather than the basement rock input. The interaction effects of the foundation are represented by a frequency-dependent dynamic stiffness matrix defined with respect to the degrees of freedom on the structure-foundation interface. The seismic input for this idealization is in the form of acceleration time-histories of the free-field motion; the response spectrum method of analysis is not applicable. This method is currently used in the analysis of gravity dams and free-standing intake towers when the foundation material can be assumed homogeneous.

(3) Soil-pile-structure earthquake excitation. Unlike the gravity dams and intake towers, lock structures may be supported on pile groups embedded in nonhomogeneous soil media. In such cases, the soil-pile-structure interaction significantly affects the earthquake response of the structure and piles and should be considered in the analysis. Figure 2-7 schematically presents two methods for the earthquake analysis of structures founded on the soil-pile foundations (Wass and Hartmann 1984). In the direct method illustrated in Figure 2-7a, the piles and the soil up to the transmitting boundaries are modeled as part of the structure. The nonlinear soil behavior may be represented by the equivalent linear method (Seed and Idriss 1969). The seismic input in the form of acceleration time-histories is applied at the rock basement (rock-soil interface), and the earthquake response of the structure and the pile forces are determined. Alternatively, the analysis may be performed in two steps consisting of the kinematic and inertial parts, with the total motion a divided into a_k and a_i caused by kinematic and inertial interactions, respectively, as shown in Figures 2-7b and 2-7c. First the kinematic interaction is evaluated using the same model employed in the direct method, except that the mass of the structure is set to zero (M = 0). This analysis provides the ground motions a_k at the structure-soil interface, the required seismic input for the subsequent dynamic analysis for the inertial-interaction effects. The dynamic stiffness matrix of the soil-pile foundation needed for the inertial interaction analysis is also determined from the analysis of the same model employed in the kinematic interaction analysis. However, the resulting dynamic stiffness (or impedance function) for the soil-pile region is a complex valued matrix that requires solution in the frequency domain. The 2-D direct method and kinematic interaction analysis described above have been used for the analysis of pile foundation with backfill soils using the FLUSH program (Olmsted Locks and Dam, Design Memorandum No. 7, U.S. Army Engineer District, Louisville 1992). These analyses also provide response spectrum seismic input at the pile tips required for performing 3-D rigid-cap pile-group dynamic analysis using the Computer-Aided Structural Engineering (CASE) computer program, X0085 (CPGD), Dynamic Analysis of Pile Groups.

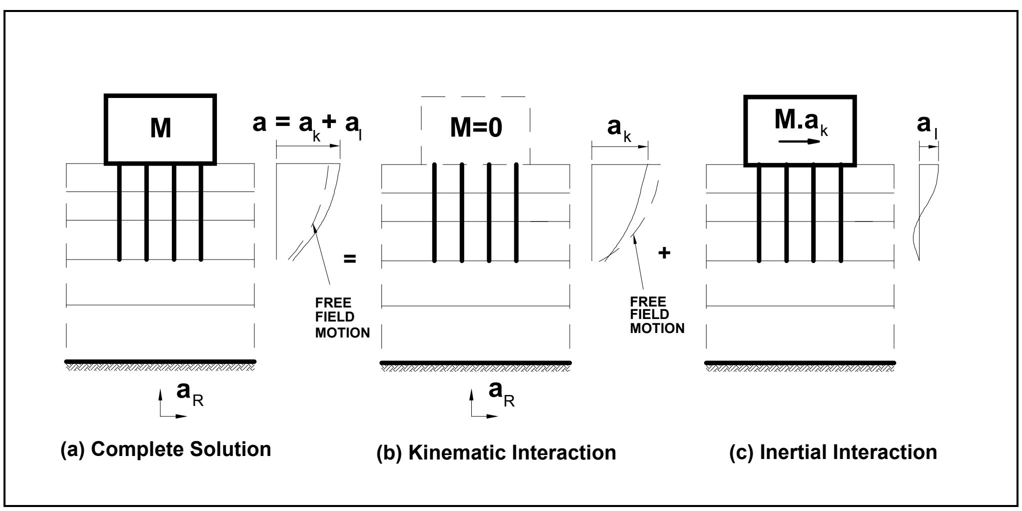


Figure 2-7. Schematic of earthquake response analysis for soil-pile-structure interaction

c. Three-dimensional (3-D) models. 3-D finite-element models are used to analyze hydraulic structures with complex geometry or nonuniform loading. Such structures include the arch dams, inclined intake towers supported on the abutment foundations, irregular free-standing intake towers with significant torsional behavior, gravity dams built in narrow canyons, and certain lock monoliths with complicated components and loading conditions. Arch dams must be treated as 3-D systems consisting of the concrete arch, foundation rock, and the impounded water (Figure 2-8). The inclined intake towers should be treated as 3-D structures to account for not only their complicated geometry and torsional behavior, but also for ground motions that must be applied at the tower base and along the entire tower-abutment interface. The irregular free-standing towers exhibiting dominant torsional modes of vibration should also be analyzed using 3-D models. Gravity dams built in narrow canyons are another example requiring 3-D treatment, because the customary assumption that dam monoliths behave independently is no longer valid—the movements of each monolith under these conditions are restrained by the adjacent ones, causing torsional moments or twists that would affect the manner in which the loads are distributed. The pile-founded lock structures with complicated geometry and structural components usually feature complicated soil-pile-structure interaction, which may require 3-D treatment. The specification of earthquake input for analysis of 3-D structures depends on the level of sophistication and capabilities used in modeling the dynamic behavior of the structure. The basic procedures are based on the general concepts described by Clough et al. (1985). These are summarized in the following paragraphs.

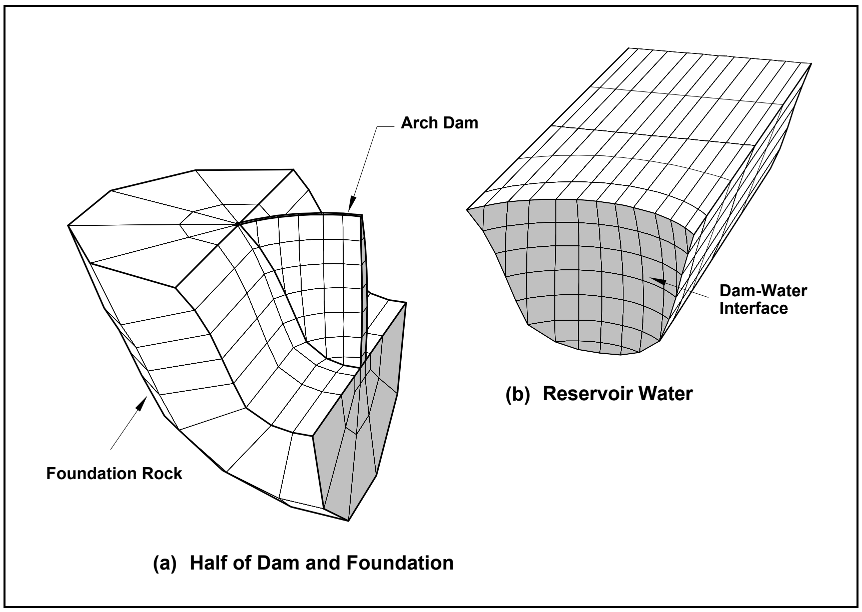


Figure 2-8. 3-D model of arch dam-water-foundation system

(1) Standard base input model. It is assumed that 3-D structures built on rock sites are supported by a large volume of deformable rock, which in turn is supported by a rigid boundary. The seismic input in the form of response spectra or acceleration time-histories is defined as the motion of this rigid base, but it should be noted that the motions applied to the rigid base differ from the free-field motions recorded at ground surface.

(2) Massless foundation rock model. An improved version of the model described in (1) above is obtained by neglecting the mass of the deformable foundation region. In this case no wave propagation takes place through the foundation rock; thus the prescribed motions at the rigid base are directly transmitted to the structure interface. With this assumption it is reasonable to use the earthquake motions

recorded at the ground surface as the rigid base input as for the 2-D analysis in Figure 2-6a. This procedure is commonly used in the practical analysis of 3-D structures built on rock sites. GDAP (Ghanaat 1993) and ADAP-88 (Fenves, Mojtahedi, and Reimer 1989) and other arch dam analysis programs commonly use this type of foundation model.

(3) Deconvolution base rock input model. In this approach the recorded free-field surface motions are deconvolved to determine the motions at the rigid base boundary. The deconvolution analysis is performed on a horizontally uniform layer of deformable rock or soil deposits using the one-dimensional wave propagation theory. For the soil sites, however, the strain-dependent nature of the nonlinear soil should be considered. The resulting rigid base motion is then applied at the base of the 3-D foundation-structure system, in which the foundation model is assumed to have its normal mass as well as stiffness properties. This procedure permits the wave propagation in the foundation rock, but requires an extensive model for the foundation rock, which computationally is inefficient.

(4) Free-field input model. A more reasonable approach for defining the seismic input would be to apply the deconvolved rigid base motion to a foundation model without the structure in place and to calculate the free-field motions at the interface positions, where the structure will be located. These interface free-field motions would be used as input to the combined structure-foundation model, which employs a relatively smaller volume of the rock region. It should be noted that the resulting seismic input at the interface varies spatially due to the scattering effects of canyon walls (in the case of arch dams) in addition to the traveling wave effects that also take place in the relatively long structures, even when the contact surface is flat. In either case, the computer program used should have capabilities to permit multiple support excitation. The application of this procedure has not yet evolved to practical problems.

(5) Soil-pile-structure interaction model. The seismic input for 3-D structures supported on pile foundations may be evaluated using a 3-D extension of the procedure discussed in b(3) above. However, the soil-pile-structure interaction analysis should also consider the inclined propagating body and surface waves if the structure is relatively long and is located close to a potential seismic source, or if it is supported on a sediment-filled basin. In particular, long-period structures with natural periods in the predominant range of surface waves should be examined for the seismic input that accounts for the effects of surface waves. One limiting factor in such analyses is the maximum number of piles that can be considered in the analysis of structures on a flexible base. For example a pile-founded lock structure may include a monolith having more than 800 piles. 3-D soil-structure interaction analysis programs such as SASSI (Lysmer et al. 1981) with pile groups analysis capability may not be able to handle such a large problem without some program modifications or structural modeling assumptions that could lead to a reduced number of piles for the idealized monolith.

2-8. Dynamic Analysis Procedures

The idealized model of structures and the prescribed earthquake ground motions are used to estimate the dynamic response of structures to earthquakes. The dynamic analysis is performed using the response spectrum or time-history method. The response spectrum method is usually a required first step in a dynamic analysis for the design and evaluation of hydraulic structures. In many cases it suffices for the structures located in low seismic hazard regions. It is also the preferred design tool, because the maximum response values for the design can be obtained directly from the earthquake response spectrum. However, the response spectrum procedure is an approximate method for calculating only the maximum response values and is restricted to the linear elastic analysis. The time-history method, on the other hand, is applicable to both linear elastic and nonlinear response analyses and is used when the time-dependent response characteristics or the nonlinear behavior is important, as explained later.

a. Modal analysis procedure. The modal superposition method is used to compute the earthquake response of structures within their linear elastic range of behavior. This procedure is especially applicable to the majority of Corps of Engineers hydraulic structures that are designed to remain essentially elastic when subjected to the medium intensity ground motions, such as the OBE. The modal analysis is also used for the MDE excitation, except that the computed linear elastic response is permitted to exceed the concrete cracking and yield stress levels for a limited amount in order to account for energy absorption of the structure. As illustrated in Figure 2-9, the primary feature of the modal analysis is that the total response of a structure is obtained by combining the response of its individual modes of vibration calculated separately. Furthermore, only the response in the first few modes need be calculated, because the response of structures to earthquakes is essentially due to the lower modes. The response of each individual mode is computed from the analysis of an SDOF system, according to the procedures described in the following paragraphs.

(1) Simplified Response Spectrum Analysis. The simplified response spectrum analysis (SRSA) is used for dynamic analysis of structures for which a simplified model of the types described in paragraph 2-7a can be developed. Whenever possible, this approximate analysis should be attempted to provide a preliminary estimate of the seismic response, as well as a basis for comparison with the results of a more refined analysis. The SRSA is normally employed for the analysis of structures whose dynamic behavior can be represented by an equivalent SDOF system. The maximum response of an idealized structure by the SRSA procedure is estimated as follows:

(a) Design response spectrum. For a preliminary analysis the standard response spectra described in paragraph 2-5a(1) should be used when a site-specific response spectrum does not exist.

(b) Natural frequencies and vibration mode shapes. Use the standard simplified procedures (Fenves and Chopra 1986, Goyal and Chopra 1989) to calculate the fundamental natural period and mode shape for the nonoverflow gravity dam sections and the regular intake towers. For other structures idealized by an equivalent SDOF system, the fundamental frequency and mode shape may be computed using the iterative methods described by Clough and Penzien (1993).

(c) Damping. Energy dissipation in the form of a damping ratio is included as part of the response spectrum curves. For the linear elastic or nearly elastic response during an OBE event, the damping value should be limited to 5 percent. For the MDE excitation, a damping constant of 7 or 10 percent may be used depending on the level of strains and the amount of inelastic response developed in the structure.

(d) Maximum modal displacement. The spectral acceleration, S_{an} (T_n ξ_n) corresponding to the nth mode (here nth mode is assumed to be the fundamental mode) period of vibration, T_n, and the specified damping ratio, ξ_n, is directly obtained from the prescribed response spectrum. The maximum modal displacement in terms of S_{an} (T_n, ξ_n) is given by:

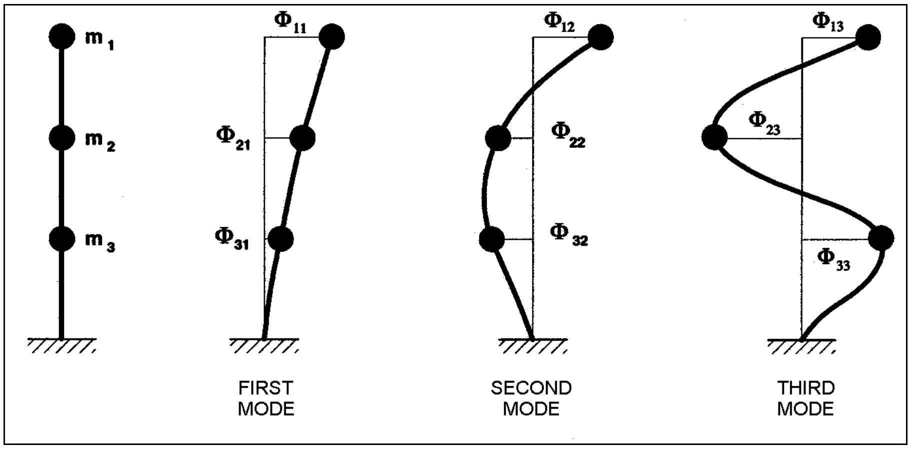
$$Y_n = \frac{L_n}{M_n \omega_n^2} S_{an}\left(T_n, \xi_n\right) \tag{2-1}$$

where

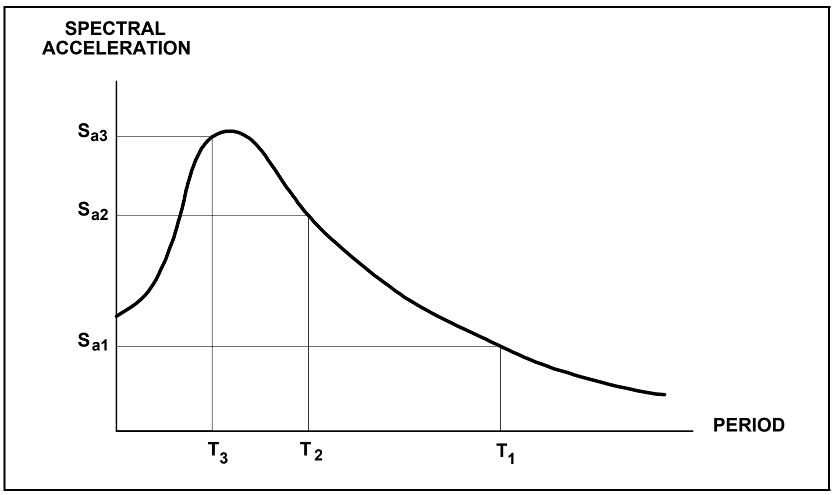
$L_n = \sum_{j=1}^{K} m_j \phi_{jn}$ is the modal earthquake-excitation factor

$M_n = \sum_{j=1}^{K} m_j \phi_{jn}^2$ is the modal mass

RESPONSE SPECTRUM ANALYSIS



(1) Compute mode shapes [ϕ_{j1}, ϕ_{j2}, ϕ_{j3}] and natural periods [T_1, T_2, T_3]



(2) Obtain spectral accelerations [S_{a1}, S_{a2}, S_{a3}] for all modes

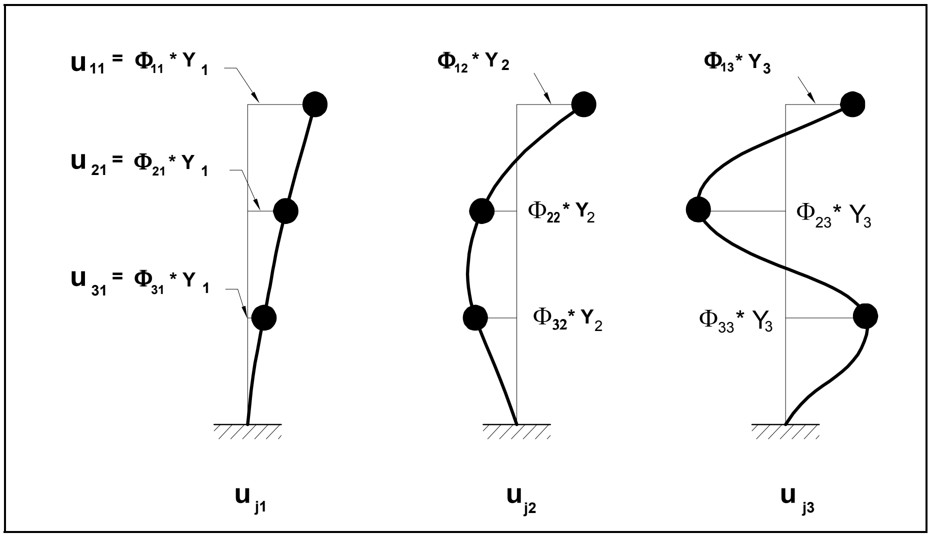
$$L_n = \sum_{j=1}^{3} \phi_{jn} m_j \quad ; \qquad M_n = \sum_{j=1}^{3} \phi_{jn}^2 m_j$$

$$PF_n = L_n / M_n$$

$$Y_n = \frac{PF_n}{\omega_n^2} \cdot S_{an} \qquad where \quad n = 1,2,3$$

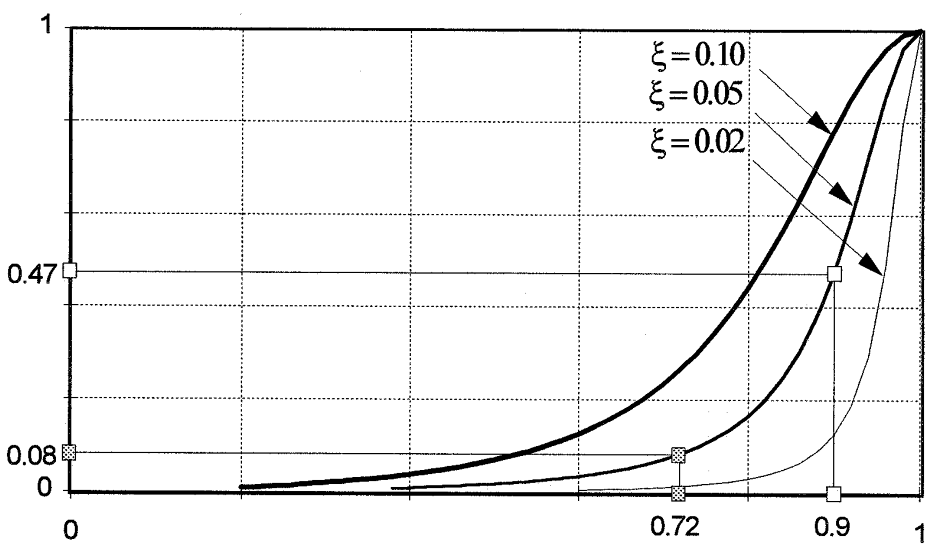
(3) Compute modal participation factor PF_n
and maximum modal response Y_n

Figure 2-9. Illustration of response spectrum mode-superposition analysis (Continued)



(4) Compute maximum response of structure in each mode from $u_n = \Phi_n * Y_n$

CROSS-MODAL COEFFICIENT, ρ_{ij}



PERIOD RATIO, $r = T_j / T_i$

(5) Obtain cross-modal coefficients ρ_{ij}

$$u_k = \left[\sum_{i=1}^{3} \sum_{j=1}^{3} u_{ki} \rho_{ij} u_{kj} \right]^{1/2}$$

(6) Compute total response u_k due to earthquake in direction *k*

Figure 2-9. (Concluded)

$\omega_n = 2\pi/T_n$ is the circular frequency

K = number of degrees of freedom in structural model

The ratio L_n/M_n is the modal participation factor PF_n indicating the degree to which mode n is excited by the ground motion.

(e) Maximum displacement. With the maximum modal displacement being computed in step (d), the maximum displacement of the structure is computed as

$$u_{jn} = \frac{L_n}{M_n \omega^2} \phi_{jn} S_{an}\left(T_n, \xi_n\right) \tag{2-2}$$

where ϕ_{jn} is the assumed or calculated mode shape of the structure.

(f) Maximum shear and moment. The shear forces and moments at sections along the height of the structure are obtained by static analysis from the equivalent lateral forces as follows:

$$f_{jn} = \frac{L_n}{M_n} m_j \phi_{jn} S_{an}\left(T_n, \xi_n\right) \tag{2-3}$$

$$V_n = \sum f_{jn} \tag{2-4}$$

$$M_n = \sum h_j f_{jn} \tag{2-5}$$

where

f_{jn} = maximum value of equivalent lateral force at the j^{th} section

h_j = associated moment arm

(2) Response spectrum modal superposition method. The estimation of maximum response of a hydraulic structure to earthquake excitation usually involves many modes of vibration, which may contribute significantly to the response. The contributions of various modes to the total displacements, forces, and stresses depend on a number of factors including the response spectrum ordinates, natural periods of vibration, and mode shapes, which in turn depend on the mass and stiffness properties of the structure. The seismic responses of such structures are further complicated by the dynamic interaction with the foundation supporting the structure and the impounded water. In general, the simplified SDOF procedures described above may not be applicable in most cases or may provide only a very crude estimate of the response. In these situations, the structure is analyzed using the response-spectrum modal-superposition method illustrated in Figure 2-9. The response-spectrum modal-superposition analysis is usually carried by standard or specialized programs following the same analysis steps described in a(1), but additional factors including the number of modes, combination of modal responses, and the effects of multiple components of earthquake input should also be considered.

(a) Number of modes. There are no guidelines for determining in advance how many modes should be included in a response spectrum analysis, because it depends on the dynamic characteristics of the structure and the response spectrum ordinates. However, the analysis should include a sufficient number of modes until the calculated response quantities are at least within 10 percent of the "exact" values. Since the "exact" response values are not known, a trial and error procedure may be adapted, in which analyses are repeated with addition of modes until it is seen that the addition of modes does not significantly affect the results. Alternatively, it may be demonstrated that the participating effective modal masses are at least within 90 percent of the total mass of the structures.

(b) Combination of modal responses. The response spectrum analysis procedure described above provides only the maximum response in each mode of vibration. The response quantities of interest, such as the peak displacements, element stresses, element forces, and moments, evaluated for each significant mode of vibration should be combined to obtain the total response of the structure. Since modal responses do not occur at the same time during the earthquake excitation, they should be combined using the complete quadratic combination (CQC) or the square root of the sum of the squares (SRSS) method described below.

(c) CQC method. The CQC modal combination method (Wilson, Der Kiureghian, and Bayo 1981) is based on random vibration theory and can be used in the response spectrum analysis if the duration of the strong motion portion of the earthquake shaking is several times longer than the fundamental period of the structure and if the design response spectrum ordinates vary slowly over a wide range of periods that include the dominant modes of the structure. Both conditions are easily met for short-period hydraulic structures and smooth design response spectra with 5 percent damping or more. The CQC formula for the maximum combined displacements u_k to an earthquake in direction k is given by

$$u_k = \left[\sum_{i=1}^{N} \sum_{j=1}^{N} u_{ki} \rho_{ij} u_{kj} \right]^{1/2} \tag{2-6}$$

where u_{ki} and u_{kj} are the maximum modal displacements corresponding to the vibration modes i and j, respectively, and N is the number of modes. The cross-modal coefficients ρ_{ij} for the above two conditions and for the constant modal damping ξ are expressed by

$$\rho_{ij} = \frac{8\xi^2 (1+r) r^{3/2}}{(1-r^2)^2 + 4\xi^2 r (1+r)^2} \tag{2-7}$$

where r is the ratio of natural period of j^{th} mode, T_j, to the natural period of the i^{th} mode, T_i. As illustrated in Figure 2-9, this equation indicates a significant interaction between closely spaced modes, especially at high damping values.

(d) SRSS method. For the structures for which the modal periods are well separated, ρ_{ij} approaches zero (for $i \neq j$), and the CQC method degenerates into the familiar SRSS method. The maximum total response for a single earthquake response spectrum in direction k is then given by

$$u_k = \left[\sum_{i=1}^{N} u_{ki}^2 \right]^{1/2} \tag{2-8}$$

The SRSS method leads to conservative results for the well-separated vibration modes, but is inappropriate when they are closely spaced, because it ignores the contribution due to the cross-modal terms.

(e) Calculation of section forces and moments. The output of most computer response-spectrum analyses usually includes nodal displacements and element stresses only. The section forces and moments required for the design of structures are not readily available, except when beam elements are used to idealize the structure. For nonbeam elements, the combined maximum stress values have no sign, and thus evaluation of section forces from these stresses, if not interpreted properly, may lead to incorrect results. Like the element stresses, section forces and moments should first be evaluated for the individual modes and then combined by the CQC or SRSS method to obtain the maximum shears, thrusts, and moments at a specified cross section.

(f) Combining for multicomponent earthquake input. Three-dimensional structures are analyzed for three orthogonal components of the earthquake ground motions applied in two horizontal and vertical direction. The maximum response quantity of interest due to each component of the earthquake ground motion is estimated separately as described above. The maximum responses due to all three components of the earthquake ground motion are then combined according to the SRSS method:

$$u = \left[\sum_{k=1}^{3} u_k^2 \right]^{1/2} \tag{2-9}$$

The SRSS combination of the multicomponent earthquake responses can be used with either the SRSS or CQC modal combination method. With the SRSS modal combination, the summation for multi-component input can be performed before or after combination of the modal responses, but with the CQC method it should always be applied after the modal responses have been combined. Regular intake towers with a circular or rectangular cross section are usually analyzed for two horizontal components of the ground motion and not the vertical. For such towers in addition to the SRSS method, the peak value of any resultant response quantity due to the combined gravity and two horizontal components of ground motion may also be obtained from the largest of the values given by the following equations:

$$R = R_0 \pm R_x \pm \alpha R_y \tag{2-10}$$

$$R = R_0 \pm \alpha R_x \pm R_y \tag{2-11}$$

where R_x is the peak response due to the x-component; R_y the y-component of horizontal ground motion; and R_0, gravity loads. Equations 2-10 and 2-11 are usually used in conjunction with the standard response spectra, and the SRSS method is employed with the site-specific response spectra. The value of α for circular towers is taken equal to 0.40 and for rectangular towers equal to 0.3.

(g) Interpretation of analysis results. The basic results of response spectrum analysis consist of the maximum nodal displacements and element stresses (or forces). As discussed previously, these maximum responses are estimated by combining responses from individual modes and multicomponent input. The resulting dynamic responses obtained in this manner have no sign and may be interpreted as positive or negative. For example, the maximum element dynamic stresses σ_d are assumed to be either tension (positive) or compression (negative). Furthermore, the maximum values associated with each response quantity are not concurrent and usually occur at different instants of time. Thus static equilibrium checks cannot be performed to validate the results. Most computer programs used to perform response spectrum analysis do not compute section thrusts, shears, and moments for elements other than beam elements. To obtain section moments and forces from the computed stress results, the analyst should assign stress signs that would produce the correct stress distribution across a specified section. This is done by a care-ful examination of the shape of the predominant response modes, from which the actual deflected shape of a member and the associated stress distributions can be predicted. As discussed previously, a better approach is first to compute modal section forces and moments from the modal stresses, and then combine them using the CQC or SRSS method.

(h) Combining static and dynamic stresses. For the evaluation of earthquake performance of hydraulic structures, the response-spectrum estimate of the dynamic stresses σ_d should be combined with the effects of the static loads σ_s. Since response spectrum stresses have no sign, combination of static and dynamic stresses should consider dynamic stresses to be either positive or negative, leading to the maximum values of the total tensile or compressive stresses:

$$\sigma_{max} = \sigma_s \pm \sigma_d \tag{2-12}$$

It should be noted that only the similarly oriented components of σ_s and σ_d can be combined.

(3) Time-history method. The linear response of structures to earthquakes can also be computed using the time-history method of analysis. In the time-history analysis normally the acceleration time-histories are used as the seismic input. Procedures for developing acceleration time-history input consistent with the design response spectrum are described in Chapter 3. The idealized structural models used in the time-history analysis are essentially identical to those described previously for the response spectrum analysis. The response history is computed using a step-by-step numerical integration procedure applied either to the original equations of motion (direct method) or to the transformed equations in modal coordinates (mode superposition) (Bathe and Wilson 1976). In the more efficient mode superposition approach, first the response history for each mode is evaluated at each integration time-step, and then the modal response histories for all significant modes of vibration are combined to determine the dynamic response of the structure.

(4) Need for time-history analysis. Linear time-history analysis is required when the results of response spectrum analysis indicate that the computed maximum total stresses (or forces) exceed the allowable values, or when special conditions exist. The time-history analysis is performed to estimate the deformations and stresses (or forces) more accurately by considering the time-dependent nature of the dynamic response to earthquake motions. The results of such analysis serve to demonstrate the general behavior of the dynamic response, and combined with rational interpretation and judgment can provide a preliminary estimate of the level of inelastic behavior. Most Corps hydraulic structures are designed essentially to respond within their linear range when subjected to low to moderate intensity earthquakes. For this level of ground motions, the linear time-history analysis provides satisfactory results. For major earthquakes it is probable that the elastic capacity of the mass concrete would be exceeded, and some cracking and crushing of the concrete and yielding of reinforcing steels could occur. Prediction of the actual response and estimation of the expected damage and inelastic behavior under severe earthquakes can be evaluated only using a more complicated nonlinear analysis. However, linear analysis can still be very valuable for a preliminary assessment of the damage and the level of postelastic response and can help to decide whether a nonlinear analysis should be performed. As part of this evaluation, the results of linear analysis for the URC hydraulic structures should be examined in a systematic manner to identify the extent of overstressed regions at any particular point in time, to produce plots showing time-histories of stresses and other response quantities of interest, and to determine statistics on the number of stress cycles exceeding the allowable values and the corresponding excursions of these stress cycles beyond the specified limits.

b. Nonlinear time-history.

(1) Need for nonlinear analysis. A nonlinear time-history analysis may be necessary when the results of a linear analysis show that the structure could suffer significant damage during a major earthquake.

Minor local damages have little effect on the overall integrity of the structure and can still be evaluated by proper interpretation of the results of linear analysis. However, when the calculated tensile stresses (or forces) are significantly greater than the tensile strength of the concrete (or section capacity) over a large region and are repeated several times during the earthquake excitation, severe cracking of the concrete, joint slippage, and yielding of reinforcements can be expected. Under these conditions, the dynamic behavior of the structure is drastically different from the linear response, and a valid estimate of the damage is possible only if a true nonlinear performance is incorporated in the analysis.

(2) Realistic nonlinear analysis. A reasonable nonlinear analysis should take into account all sources of nonlinearity that contribute significantly to the nonlinear response behavior. The damage caused by earthquake shaking is normally associated with significant loss in the structural stiffness resulting from the concrete cracking, yielding of steel, opening of construction joints, slippage across the construction joints or cracking planes, and the nonlinear material behavior. Additional sources of nonlinearity arise from the nonlinear soil and the fractured foundation rock supporting the structure, as well as the separation of the structure and the foundation at the contact surface. At the present time, analytical techniques for a complete nonlinear earthquake analysis of hydraulic structures, including the interaction with foundation and water, are not available. Only limited aspects of the nonlinear behavior such as the contraction joint opening in arch dams, tensile cracking of gravity dams, sliding of blocks, and approximate postelastic analyses have been considered in practice. A realistic nonlinear analysis for the seismic safety evaluation of hydraulic structures depends on a great deal of new developments in the following topics: definition of seismic input, identification and specification of significant nonlinear mechanisms (joint opening and sliding, tensile cracking of the concrete, yielding and slippage of reinforcing steel, nonlinear material behavior under cyclic loads, etc.), development of idealized models representing the nonlinear behavior, numerical techniques and solution strategies for computing the nonlinear response, and development of criteria for acceptable performance and identification of possible modes of failure. The seismic input for a nonlinear analysis is in the form of acceleration time-histories. The key issues in developing time-histories for nonlinear analysis are duration of strong shaking, energy and pulse sequencing, special near-fault characteristics such as the source “fling,” and the number of sets of time-histories required for the analysis. The main difficulty in effective nonlinear analysis at the present is the lack of or limited knowledge on the actual nonlinear material properties of the mass and reinforced concrete under cyclic loading.

2-9. Sliding and Rotational Stability During Earthquakes

a. Sliding stability. The sliding stability evaluation of hydraulic structures under earthquake loading can be made according to the traditional static equilibrium (seismic coefficient) and permanent displacement approaches described in Ebeling and Morrison (1992). In the traditional approach, the sliding stability is expressed in terms of a prescribed factor of safety against sliding, whereas in the permanent displacement approach the structure is permitted to slide along its base but the accumulated displacement during the ground shaking should be limited to a specified allowable value.

(1) Seismic coefficient approach. In the seismic coefficient approach, the safety against sliding is determined on the basis of shear-friction factor of safety (Ebeling and Morrison 1992). The shear-friction factor of safety is defined as the ratio of the resisting to driving forces along a potential failure surface

$$FS = \frac{CA + \left(\sum N - \sum U\right)\tan\phi}{\sum V} \tag{2-13}$$

where

C = unit cohesion

A = area of base

$\sum N$ = summation of normal forces

$\sum U$ = summation of uplift forces

$\tan \phi$ = coefficient of internal friction

$\sum V$ = summation of shear or driving forces

(a) The driving forces acting on the structure include the static and seismic inertia forces due to weight of the structure and to hydrodynamic pressures. Treating the system above the failure surface as a rigid block, the inertia force associated with the mass of the structure is computed as the product of the seismic coefficient, as specified in ER 1110-2-1806, and the weight of the block. Similarly, the product of the seismic coefficient and the added mass of water moving with the structure produces inertia force due to the hydrodynamic pressure. The added mass of water may be computed using the Westergaard method (Westergaard 1933) or the equation given by Chopra (Chopra 1967):

$$M_a = 0.54\rho h^2 \tag{2-14}$$

where ρ is the density and h is the depth of water. The motion of the structure relative to the failure surface is resisted by the shear strength mobilized between the structure and the surface by the friction and cohesion, as shown in Equation 2-13. For example, the shear friction factor of safety for sliding of the gravity dam shown in Figure 2-10 is given by

$$FS = \frac{CA + (W - U)\tan\phi}{\pm H_s + H_d + \left(\frac{W}{g}\right)a} \tag{2-15}$$

where

W = weight of dam

H_s = hydrostatic force

H_d = hydrodynamic force (i.e., $M_a \, . \, a$)

g = gravitational acceleration

a = ground motion acceleration or some fraction thereof

The ± sign is for sliding in the downstream or upstream direction with the plus sign indicating downstream.

(b) When the earthquake forces are included in the sliding stability analysis, the calculated factor of safety against sliding may become less than one. A factor of safety of less than one indicates a transient sliding. The sliding is assumed to occur for as long as the ground acceleration is greater than the critical value required for the driving force to exceed the resistance. However, due to the oscillatory nature of the earthquake ground motion, the sliding displacement is expected to be limited but could lead to excessive permanent displacements.

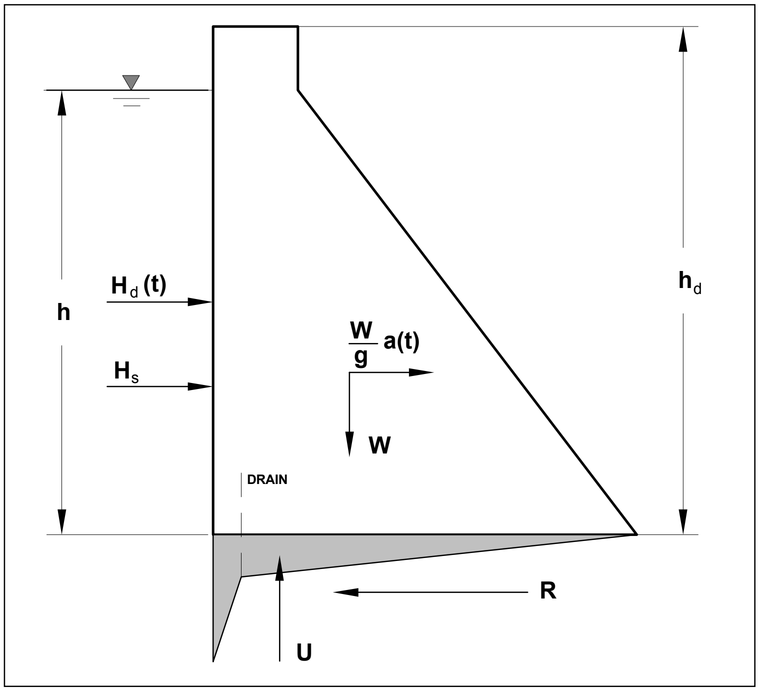


Figure 2-10. Forces acting on gravity dam monolith

(2) Permanent displacement approach. The traditional sliding stability criteria described above were developed for unrealistically small seismic forces based on a seismic coefficient of 0.1 or less. The factor of safety against sliding required by the traditional approach may not be attainable for larger seismic forces representative of the moderate- to high-intensity earthquake ground motions. In such cases, the sliding may occur but it takes place only during a short period of time associated with the acceleration cycles exceeding a critical acceleration, a_c, and diminishes during the remainder of these cycles when the acceleration is less than a_c and the relative velocity between the structure and the base is zero. Treating a gravity dam monolith as a rigid body supported on horizontal ground, and assuming that the motion of the dam relative to the ground is resisted by the friction between the dam and the ground surface, the critical acceleration a_c is given by (Chopra and Zhang 1991)

$$\frac{a_c}{g} = \frac{1}{W + W_{a0}}\left[\mu_s\left(W - U\right) \pm H_s\right] \tag{2-16}$$

where μ_s is the coefficient of static friction, and W_{a0} is the weight of water which represents the hydrodynamic force. The ± sign in this equation is for sliding in the upstream or downstream direction. It is apparent from Equation 2-16 that the critical acceleration required to slide the dam downstream is smaller than that needed to move the dam upstream. Similarly, the critical acceleration a_c necessary to initiate sliding in other hydraulic structures can be derived by considering the equilibrium of forces involved in each particular structure. Knowing the critical acceleration a_c, the permanent sliding displacements can be estimated using the Newmark's rigid block model (Newmark 1965). According to Newmark's concepts, also discussed by Chopra and Zhang (1991), the upper bounds for permanent displacements of the sliding rigid mass subjected to earthquake ground motion with peak velocity v_m and peak acceleration a_m can be estimated from :

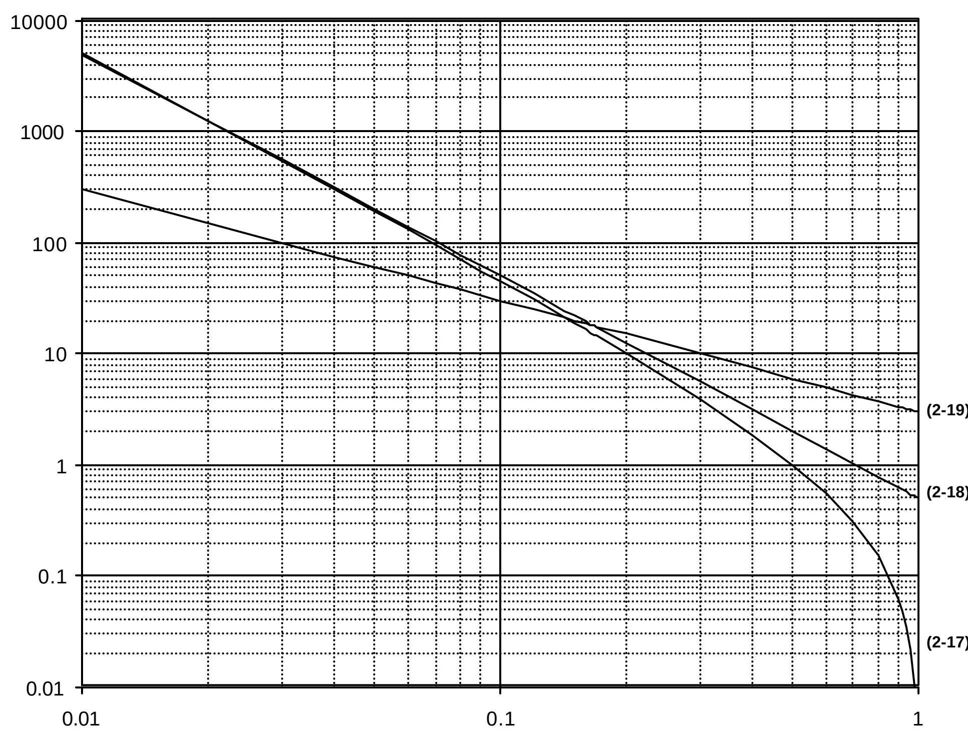
$$s_m = \frac{v_m^2}{2a_c}\left(1 - \frac{a_c}{a_m}\right)\frac{a_m}{a_c} \tag{2-17}$$

$$s_m = \frac{v_m^2}{2a_c} \cdot \frac{a_m}{a_c} \tag{2-18}$$

$$s_m = \frac{v_m^2}{2a_c} 6 \tag{2-19}$$

These equations, plotted in nondimensional form in Figure 2-11, show that Equation 2-18 provides more conservative values than Equation 2-17, and Equation 2-19 is intended for systems with the small values of a_c/a_m. The portion of the curve for each equation where the equation is recommended for use is a thick line. Newmark's model provides an easy means for approximate estimation of the upper bounds for permanent sliding displacements, but it is based on certain assumptions that ignore the true dynamic response behavior of the sliding. More accurate estimates of the sliding displacements can be made from the response history analysis proposed by Chopra and Zhang (1991).

Permanent Sliding Displacement * (a_m/v_m^2)



a_c/a_m = Critical Acceleration/Peak Ground Accleration

Figure 2-11. Newmark's upper bounds for permanent sliding displacement (Chopra and Zhang (1991), courtesy of Earthquake Engineering Research Center, University of California, Berkeley)

b. Rotational stability. Hydraulic structures subjected to large lateral forces produced by major earthquakes may tip and start rocking when the resulting overturning moment becomes so large that the structure breaks contact with the ground.

(1) Intake towers. For an intake tower idealized as a nearly rigid or flexible equivalent SDOF system (Figure 2-12), the tipping occurs when the overturning moment exceeds the resisting moment due to the weight of the structure. This condition is expressed by:

$$mS_a h > mgb \tag{2-20}$$

$$S_a > g\,(b/h) \tag{2-21}$$

where

m = mass of structure

S_a = spectral acceleration of the earthquake ground motion

h = one-half height of structure

b = one-half base width of structure

Similar expressions can also be derived for other hydraulic structures, except that the moments due to hydrostatic and hydrodynamic forces should be included ((2) below). In both cases it is assumed that the structure is not bonded to the ground, but it may be keyed into the soil with no pulling resistance. It should be noted that the structure will eventually overturn if the moment $M > mgb$ is applied and sustained, where mgb represents the resisting moment due to the weight of the structure. However, under earthquake excitation large overturning moments occur for only a fraction of a second in each cycle, with intermediate opportunities to unload. By comparing the earthquake average energy input with the required average energy for overturning the structure, Housner provided the following relationship as a criterion for the rotational stability of a rocking structure (Housner 1963):

$$\alpha = S_v \sqrt{\frac{mr}{gI_0}} \tag{2-22}$$

where

α = an angle defined in Figure 2-12

S_v = spectral velocity of the earthquake ground motion

m = mass of structure

r = radial distance from the center of gravity to tipping edge

I_0 = moment inertia about the corner

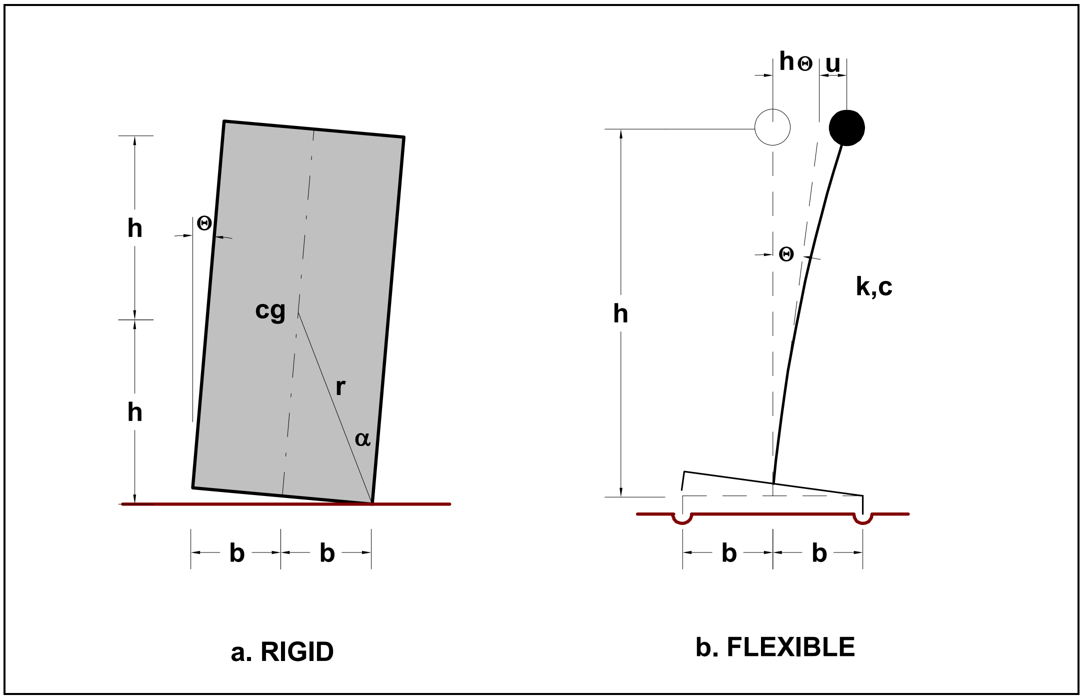


Figure 2-12. Rigid block and SDOF models for rigid and flexible structures

Based on the average energy formulation used, this equation is interpreted as stating that for a given spectral velocity S_v, a block having an angle of α given by Equation 2-22 will have approximately a 50 percent probability of being overturned (Housner 1963). For slender structures such as intake towers, Equation 2-22 can be approximated by

$$\alpha = \frac{S_v}{\sqrt{gr}} \tag{2-23}$$

By combining Equations 2-21 and 2-23 and using the relationships among the spectral acceleration, velocity, and displacement, Scholl (Applied Technology Council (ATC) 1984) found that consideration of one spectral parameter alone as the earthquake demand is not sufficient for evaluating overturning and suggested the following relationships:

$$S_d = b \quad \text{when} \quad S_a = g\frac{b}{h} \tag{2-24}$$

These equations show that when S_a is just sufficient to cause tipping, the structure will start rocking, but its displacement approximated by spectral displacement S_d must reach the value of b before it can overturn. These equations also demonstrate why larger structures such as buildings do not overturn during earthquakes, whereas smaller rigid blocks having the same aspect ratios are expected to overturn. This is because, in general, S_d is never large enough to tip over a building, but it can approach the one-half base width (i.e., b) of smaller rigid blocks such as tombstones.

(2) Gravity dams. A preliminary study of the rotational stability of a gravity dam may be carried out as described by Chopra and Zhang (1991). The dam is assumed to be rigid and subjected to both the horizontal and vertical components of earthquake ground motion. The dam starts tipping in the

downstream direction if the overturning moment due to upstream ground acceleration and water pressures exceeds the restoring moment (Figure 2-13) as follows:

$$M|a_x(t)|y_c + M_d(t) + \frac{1}{3}hH_s + U(b - b_U) \geq M[g + a_y(t)](b - x_c) \tag{2-25}$$

where parameters are shown in Figure 2-13 and $M_d(t)$ is the moment due to hydrodynamic pressure given by

$$M_d(t) = -M_{d0}a_x(t) + \frac{1}{6}\rho h^3 a_y(t) \tag{2-26}$$

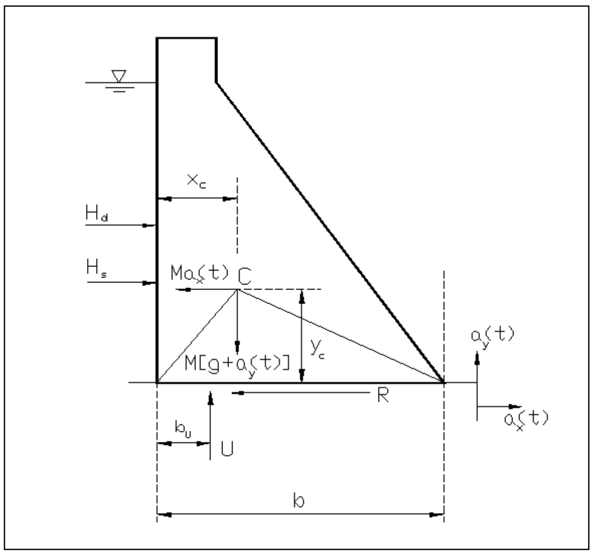


Figure 2-13. Rigid gravity dam on horizontal ground

The first term, $M_{d0} = \int_0^h p_0(y)y\,dy$, is moment due to hydrodynamic pressure generated by the horizontal ground motion. The pressure $p_0(y)$ may be obtained using either the Westergaard (1933) or Chopra (1967) formula. The second term in Equation 2-26 is the moment due to hydrodynamic pressure, $\rho(h - y)$ $a_y(t)$, produced by the vertical motion. Substituting Equation 2-26 into 2-25 gives the critical upstream acceleration a_c required to initiate downstream tipping of the dam about its toe:

$$a_c = \frac{1}{My_c + M_{d0}} \times \left\{\left[M(b - x_c) - \frac{1}{6}\rho h^3\right][g + a_y(t)] - U(b - b_u)\right\} \tag{2-27}$$

Similarly, tipping about the heel of the dam initiates when the critical downstream acceleration a_c reaches

$$a_c = \frac{1}{My_c + M_{d0}}\left\{\left(Mx_c + \frac{1}{6}\rho h^3\right)[g + a_y(t)] - Ub_u\right\} \tag{2-28}$$

When subjected to strong ground motions, a rigid dam may move with ground, slide only, rock only, and rock and slide. Comparison of critical accelerations for sliding and tipping can show which motion will start first. The parameter studies by Chopra and Zhang (1991) over a wide range of parameters indicate that downstream sliding of the dam will initiate before tipping or upstream sliding. Large downstream accelerations will usually cause upstream tipping of the dam about its heel before upstream sliding. In most cases, the more likely type of motion is the downstream sliding of the dam. The effect of rocking on the sliding motion is considered to be negligible and may be ignored in the evaluation of sliding response.

2-10. Current Practice on Use of Response Spectra for Building-Type Structures

a. General. The design requirements of the Structural Engineers Association of California (SEAOC) are the most current state of practice for the earthquake-resistant design of buildings in California. SEAOC also includes provisions for lower seismic hazard regions in California, which may be suitable for use in regions outside of California. A summary of the SEAOC's use of design response spectra and the similarities and differences between the SEAOC's recommendations and the procedures used for the hydraulic structures are described in this section.

b. Criteria for dynamic analysis. According to SEAOC's recommendations, dynamic analysis procedures should be used for the design and analysis of certain building structures. This includes buildings 73.2 m (240 ft or more) in height, except for those located in Seismic Zone 1 and standard occupancy structures in Zone 2, buildings with irregular stiffness, mass, or vertical geometry, buildings over five stories or 20 m (65 ft) in height located in Seismic Zones 3 and 4, or buildings founded on soft soils (type S4) with fundamental period of vibration greater than 0.7 sec. The dynamic analysis procedures for buildings are based on the same general concepts described for the hydraulic structures. The response spectrum analysis is the preferred method for most buildings. The time-history analysis is employed to study inelastic response characteristics or to incorporate time-dependent effects in the elastic dynamic response. However, structural modeling, design earthquakes, and acceptable level of nonlinear response for buildings are different from those for hydraulic structures.

c. Structural modeling. The obvious differences between building and hydraulic structures are the structural system and the function. While buildings are made primarily of frame systems with shear walls and braces, hydraulic structures are built as massive plain or lightly reinforced concrete monoliths to contain or retain water. The idealized model of a building usually consists of beam and column elements with the mass of the building lumped at a few selected nodes. Most regular buildings can be adequately idealized by one- or two-dimensional models. Very complex and highly irregular buildings or those with large eccentricities between the center of mass and resistance require 3-D analysis. The majority of hydraulic structures, on the other hand, are modeled as planar or 3-D models using 2-D or 3-D solid and shell elements as discussed previously. Furthermore, interactions with the water and foundation are important aspects of the dynamic characteristics of hydraulic structures that need to be included in the analysis.

d. Design response spectra. Hydraulic structures and civil works buildings are analyzed for two levels of design earthquakes (as described previously and in reference to ER 1110-2-1806, respectively). SEAOC requires a single design earthquake that as a minimum should have a 10 percent probability of exceedance in 50 years, which gives a return period of 475 years. The ground motion for buildings may be represented by normalized response spectra, site-specific response spectra, or time-histories. The normalized response spectra, shown in Figure 2-14, are permitted for the soil profiles S_1, S_2, and S_3 defined in Table 2-1. Site-specific response spectra are required if the site condition significantly differs from those used to develop the normalized shapes, or if the structure is founded on soil profile S4, or the structure is seismically isolated. The time-histories of the ground motions are usually developed for inelastic analysis and for the seismic-isolated structures. The probabilistic seismic hazard analysis is commonly used to develop site-specific response spectra for building sites in Zones 3 and 4.

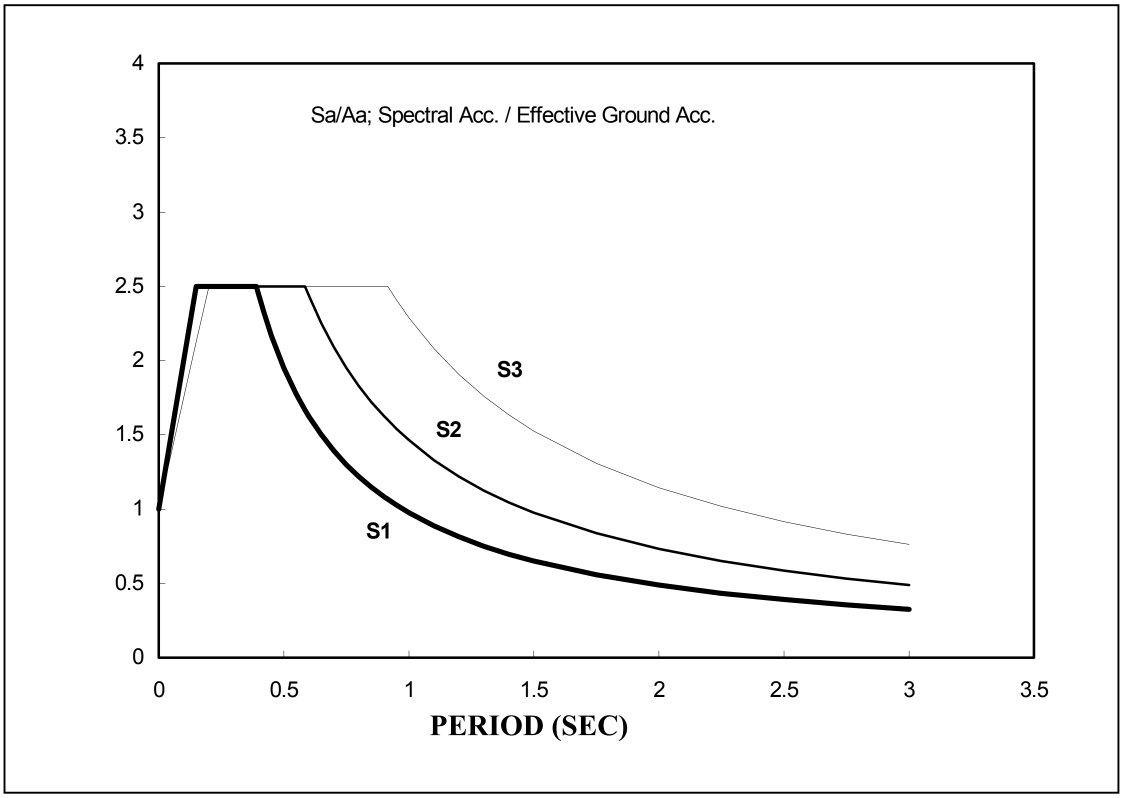


Figure 2-14. Normalized SEAOC's design response spectra

Table 2-1
SEAOC's Site Coefficients

Type	**Description**	S Factor
S1	A soil profile with either: 1 A rock-like material characterized by a shear wave velocity greater than 762 m/sec (2,500 fps) or by other suitable means of classification, or 2 Medium-stiff to stiff or medium dense to dense soil conditions where soil depth is less than 70 m (200 ft).	1.0
S2	A soil profile with predominantly medium dense to dense or medium stiff to stiff soil conditions where soil depth exceeds 70 m (200 ft) or more.	1.2
S3	A soil profile containing more than 6 m (20 ft) of soft to medium stiff clay but not more than 12 m (40 ft) of soft clay.	1.5
S4	A soil profile characterized by a shear wave velocity less than 152 m/sec (500 fps) and containing more than 12 m (40 ft) of soft clay.	2.0

1 Reprinted from SEAOC's Recommended Lateral Force Requirements and Commentary, 1996 (SEAOC 1996).

e. Nonlinear response of buildings. The design practice for buildings permits the structure to respond inelastically during a major earthquake but suffer only acceptable and predictable amounts of damage without collapse. The energy dissipated through such inelastic deformations is utilized to reduce the level of seismic design forces. According to the SEAOC's Commentary, structures designed in conformance with its recommendations should, in general, be able to

(1) Resist a minor level of earthquake ground motion without damage.

(2) Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.

(3) Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast at the building site, without collapse, but possibly with some structural as well as nonstructural damage. To achieve such performance goals, SEAOC recommends that the design forces be determined by reducing the elastic forces obtained from linear analysis by the system quality factor R_w. The structural system quality factor R_w represents the overall ductility and energy dissipation capacity of the system when strained beyond its elastic limit. The R_w values are selected according to the ability of a particular system to sustain cyclic inelastic deformations without collapse. Factors contributing to the actual selection of R_w include redundancy, reliability of as-built performance, inelastic load-deformation behavior, and changed damping and period modification with deformation. Experience indicates that buildings designed based on this procedure have exhibited adequate performance in most cases. In addition to the ductility requirement, most seismic codes require a limitation on the story drift to limit nonstructural damage during more frequent earthquakes and to ensure building stability under the major earthquakes. For tall buildings, the drift limitations may dictate an elastic design, even for MCE ground motions.

f. Inelastic design response spectra. For regular buildings, the code specified design seismic forces can be estimated from

$$V = C_s W = \frac{(S_a/g)}{R_w} \cdot W \qquad (2\text{-}29)$$

where

V = base shear

C_s = design seismic coefficient

W = weight of building

S_a = linear elastic response spectral acceleration

R_w = the structural system quality factor intended to account for ductility and energy dissipation when the structure deforms beyond the yield point

The design seismic coefficient C_s can be obtained from a reduced response spectrum that can be considered to be the inelastic design response spectrum (IDRS). However, recent statistical studies have shown that the shape of IDRS significantly differs from the shape of elastic response spectra (Miranda 1992; Krawinkler and Rahnama 1992), and is strongly influenced by the level of inelastic deformation, local site conditions, and the period of vibration. Until more data become available, the most reliable procedure for establishing an IDRS is to perform nonlinear dynamic time-history analyses of structures with different degrees of ductility ratios using the available recorded ground motions.

g. Nonlinear inelastic response of reinforced concrete hydraulic structures.

(1) Free-standing intake towers. The response of a free-standing intake tower to earthquake ground shaking is similar to that of a cantilever wall that resists earthquake forces by flexural and shear deformations. Thus, the principles of inelastic design of reinforced concrete walls are generally applicable to free-standing towers, provided that the response is consistent with the post-yield capacity of reinforced tower structure. Similar to the structural walls with limited ductility, intake towers should be designed such that flexural yielding controls the strength, the inelastic deformations, and thus the energy dissipation in the entire structure (Pauly 1986). Failure modes due to diagonal tension or diagonal compression caused by shear and sliding shear along the base of the structure should be avoided. The main source of energy dissipation should be yielding of flexural reinforcement in the region where the plastic hinges are expected to develop. Recent research at CEWES has demonstrated that a lightly reinforced rectangular intake tower possesses sufficient ductility to allow formation of a plastic hinge at the base of the tower with a limited amount of inelastic behavior. The ability of such plastic hinges to sustain the repeated cycles of inelastic demands imposed on them, the expected length of the plastic hinge, and the acceptable level of damage during major earthquakes are being investigated at CEWES. Until these have been established, preliminary design and screening evaluations for retangular intake towers should be limited to an inelastic response not greater than twice the yield deflection in accordance with the design provisions of EM 1110-2-2400.

(2) Other reinforced concrete hydraulic structures. For other reinforced concrete hydraulic structures with structural configurations and systems different from those of buildings and intake towers, postelastic analyses should be performed to establish appropriate ductility for the members so that the structure will undergo controlled levels of nonlinearity without compromising structural safety. Designing for to MDE take advantage of the ability of the structures to dissipate energy through inelastic deformations should produce uniform patterns of yielding and energy absorption. While the initial forces are distributed through the structure according to the elastic stiffness of the various members and connections, partially developed plastic hinges in the critical members will redistribute forces to the stiffer members and may not follow the load paths envisioned in the initial design. If the desired level of ductility is not reached or the forces are not distributed appropriately, then it is possible that the actual member forces may exceed the design values. In addition, the possibility that all cycles of nonlinear response (i.e., hysteretic energy demand), and not just the maximum response cycle, may cause damage should also be investigated. In other words, it is important to note that the specified ductility may be reached either once or several times during the ground shaking, and to ensure that the lightly reinforced concrete hydraulic structure has sufficient ductility to resist such repeated demands. These considerations indicate that the inelastic design of hydraulic structures requires careful attention to the actual postelastic behavior of the structure and should be done in consultation with and approved by CECW-ET.

Chapter 3
Development of Site-Specific Response Spectra for Seismic Analysis of Concrete Hydraulic Structures

Section I
Introduction

3-1. Purpose and Scope

This chapter describes procedures for developing site-specific response spectra of ground motions throughout the United States for seismic analyses of hydraulic structures. Also covered, but in less detail, are approaches for developing acceleration time-histories of ground motions. Section I provides an overview of the general approaches to developing site-specific response spectra, a brief discussion of factors influencing earthquake ground motions, and a brief discussion of differences in ground motion characteristics in different regions of the United States. This is followed in Sections II and III by descriptions of procedures for developing site-specific response spectra using deterministic and probabilistic approaches, respectively.

3-2. General Approaches for Developing Site-Specific Response Spectra

The two general approaches for developing site-specific response spectra are the deterministic and probabilistic approaches.

a. Deterministic approach.

(1) General. In this approach, often termed a deterministic seismic hazard analysis, or DSHA, site ground motions are deterministically estimated for a specific, selected earthquake, that is, an earthquake of a certain size on a specific seismic source occurring at a certain distance from the site. The earthquake size may be characterized by magnitude or by epicentral intensity. In the WUS, the practice has been to use magnitude, whereas in the EUS, both magnitude and Modified Mercalli Intensity (MMI) have been used. However, in the EUS, magnitude is increasingly being used as the measure of earthquake size, and ground motions are correspondingly being estimated using correlations with magnitude. In this manual, ground motions are estimated using relationships with magnitude. For procedures for estimating ground motions as a function of intensity, reference should be made to the state-of-the-art publication by Krinitzsky and Chang (1987).

(2) Size and location of design earthquakes. In the deterministic approach, earthquake magnitude is typically selected to be the magnitude of the largest earthquake judged to be capable of occurring on the seismic source, i.e., MCE. The selected earthquake is usually assumed to occur on the portion of the seismic source that is closest to the site (an exception is the “random earthquake” analysis described in Section II and Appendix D). After the earthquake magnitude and distance are selected, the site ground motions are then estimated using ground motion attenuation relationships or other techniques. Procedures for deterministically estimating earthquake ground motions are described in Section II.

b. Probabilistic approach. In the probabilistic approach, often termed a probabilistic seismic hazard analysis, or PSHA, site ground motions are estimated for selected values of the probability of ground motion exceedance in a design time period or for selected values of annual frequency or return period for ground motion exceedance. As an example, ground motions could be estimated for a 10 percent probability of exceedance in 100 years or for a return period of 950 years. The probabilistic approach thus provides an expression of potential earthquake loading in terms analogous to those used for other environmental loads in civil engineering design such as wind and flood loading. A probabilistic ground motion assessment incorporates the frequency of occurrence of earthquakes of different magnitudes on the seismic sources, the uncertainty of the earthquake locations on the sources, and the ground motion attenuation including its uncertainty. Section III describes the procedures for probabilistically estimating earthquake ground motions.

3-3. Factors Affecting Earthquake Ground Motions

a. General. As stated in paragraph 1-8, It has been well-recognized that earthquake ground motions are affected by earthquake source conditions, source- to-site transmission path properties, and site conditions. The source conditions include the stress drop, source depth, size of the rupture area, slip distribution (amount and distribution of static displacement on the fault plane), rise time (time for the fault slip to complete at a given point on the fault plane), type of faulting, and rupture directivity. The transmission path properties include the crustal structure and the shear-wave velocity and damping characteristics of the crustal rock. The site conditions include the rock properties beneath the site to depths of up to about 2 km, the local soil conditions at the site to depths of up to several hundred feet, and the topography of the site. In developing relationships for estimating ground motions, the effects of source, path, and site have been commonly represented in a simplified manner by earthquake magnitude, source-to-site distance, and local subsurface conditions. The important influences of these factors on ground motion are summarized below.

b. Effects of earthquake magnitude and distance on ground motions.

(1) General. The effects of earthquake magnitude and distance on the amplitude of ground motions are well known; ground motion amplitudes tend to increase with increasing magnitude and decreasing distance. However, the effects of magnitude and source-to-site distance on relative frequency content (response spectral shape) and duration of ground motions are not as well known and therefore are briefly reviewed below.

(2) Effects of magnitude. The Guerrero, Mexico, accelerograph array data illustrate the large effect of magnitude on the relative frequency content and duration of earthquake ground motions. The Guerrero array has provided recordings of rock motions over a wide range of magnitudes. Figures 3-1 and 3-2 show the accelerograms and the corresponding response spectra for six recordings selected by Anderson and Quaas (1988) to be approximately equally spaced in magnitude from the smallest event of magnitude 3.1 to the largest of magnitude 8.1. Figure 3-1 illustrates the effect of magnitude on the duration of the strong shaking part of an accelerogram; with increasing magnitude, duration rapidly increases. All events have epicenters about 25 km from the station, and all stations are on hard rock (Anderson and Quaas 1988). Figure 3-2 illustrates that the larger magnitude events have somewhat larger spectral amplitudes at high frequencies and much larger spectral amplitudes at long periods. In other words, increasing magnitude results in greatly enriched relative frequency content (higher spectral shapes) at long periods. Figure 3-3 illustrates a generalization of the effect of magnitude on response spectral shape, in this case, from the empirically based attenuation relationships for rock developed by Sadigh et al. (1993).

(3) Effects of distance. Data from the Loma Prieta earthquake of October 17, 1989, provide an example of the effect of distance on response spectral shape. Figure 3-4 shows that the spectral shape of the recordings obtained on rock during this earthquake reduce in the high-frequency range and increase in the long-period range with increasing distance. A generalization of the effect of distance on spectral

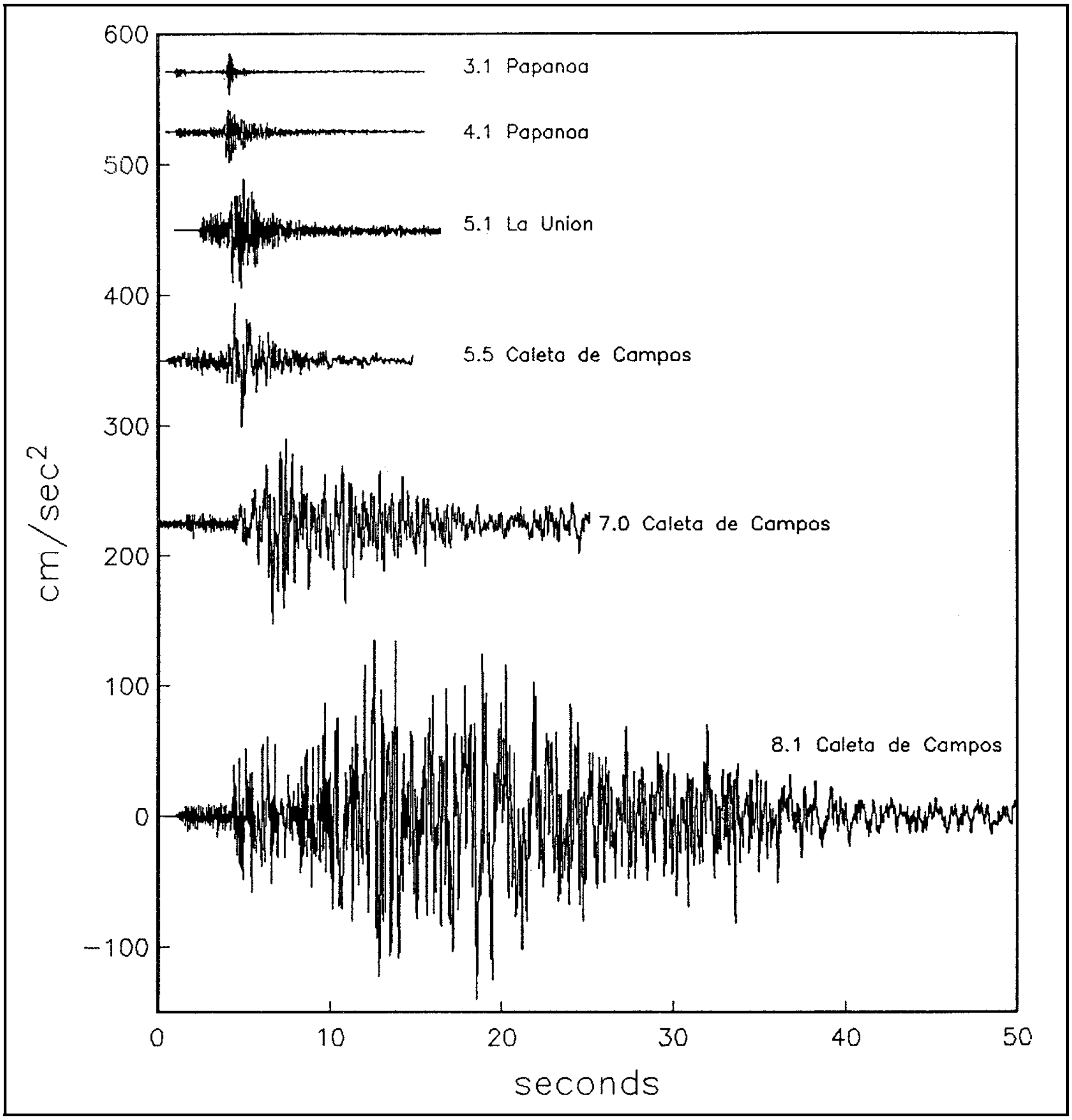


Figure 3-1. An example of accelerograms recorded in 1985 and 1986 on the Guerrero accelerograph array (Anderson and Quaas (1988), courtesy of Earthquake Engineering Research Institute, Oakland, CA)

shape, in this case from the theoretically based relationships for rock of Silva and Green (1989), is illustrated in Figure 3-5. In general, within source-to-site distances of about 50 km, the effect of distance on spectral shape is much smaller than the effect of magnitude. Similarly, although the relative duration of the strong shaking part of an accelerogram tends to increase with increasing distance (e.g., Dobry, Idriss, and Ng 1978), this effect appears to be relatively small within 50 km of an earthquake source.

(4) Special effects of near-source earthquakes. Near the earthquake source (within approximately 10 to 15 km of the source), earthquake ground motions often contain a high-energy pulse of medium-to-long-period ground motion (at periods in the range of approximately 0.5 to 5 sec) that occurs when fault

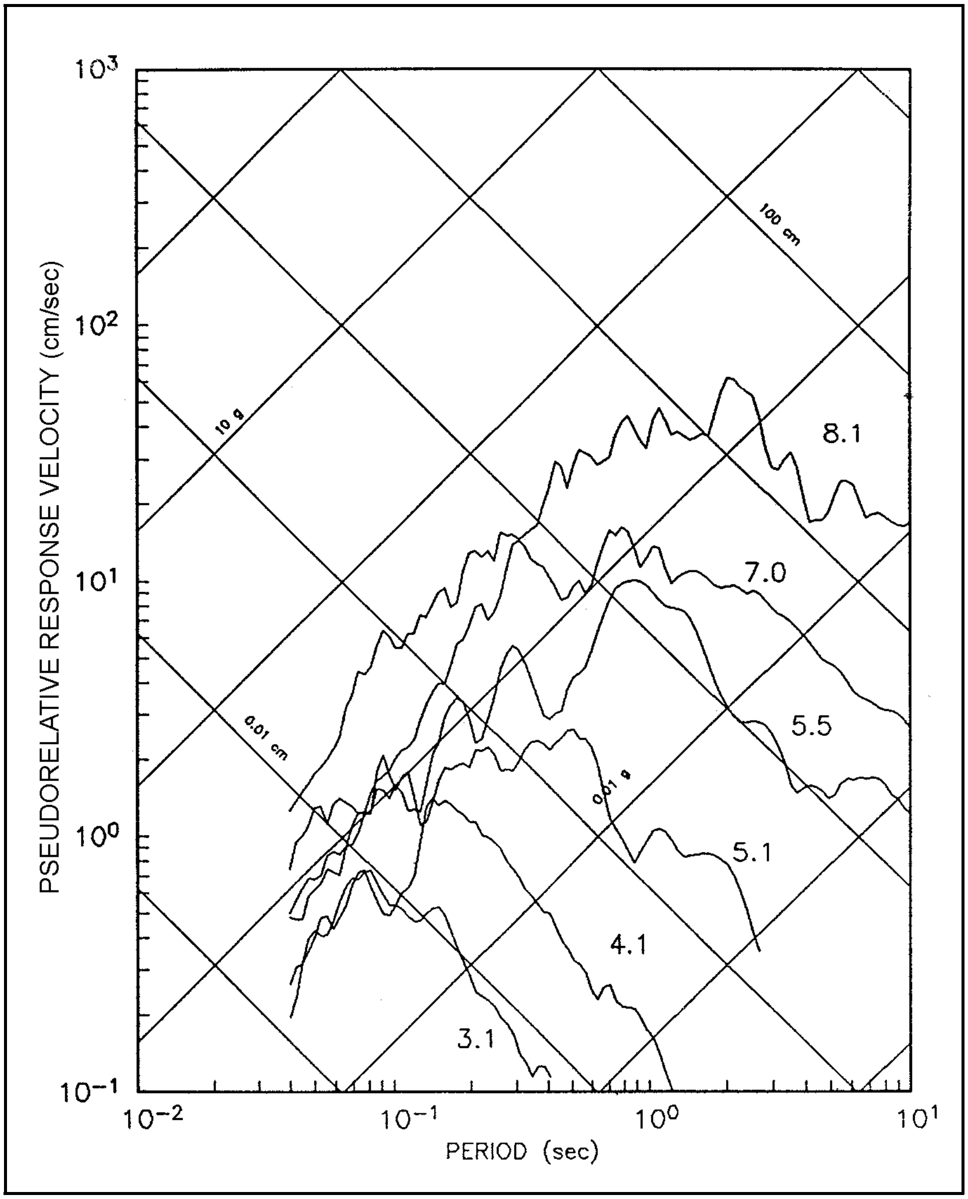


Figure 3-2. Response spectra (5 percent damped, pseudo-relative velocity) corresponding to the acceleration traces in Figure 3-1 (Anderson and Quaas (1988), courtesy of Earthquake Engineering Research Institute, Oakland, CA)

rupture propagates toward a site. It has also been found that these pulses exhibit a strong directionality, with the component of motion perpendicular (normal) to the strike of the fault being larger than the component parallel to the strike (see, for example, Sadigh et al. 1993; Somerville and Graves 1993; Somerville et al. 1997). These characteristics of near-source ground motions are illustrated by the Rinaldi recording obtained during the 1994 Northridge earthquake (Figure 3-6). These characteristics should be included in ground motion characterization for near-source earthquakes.

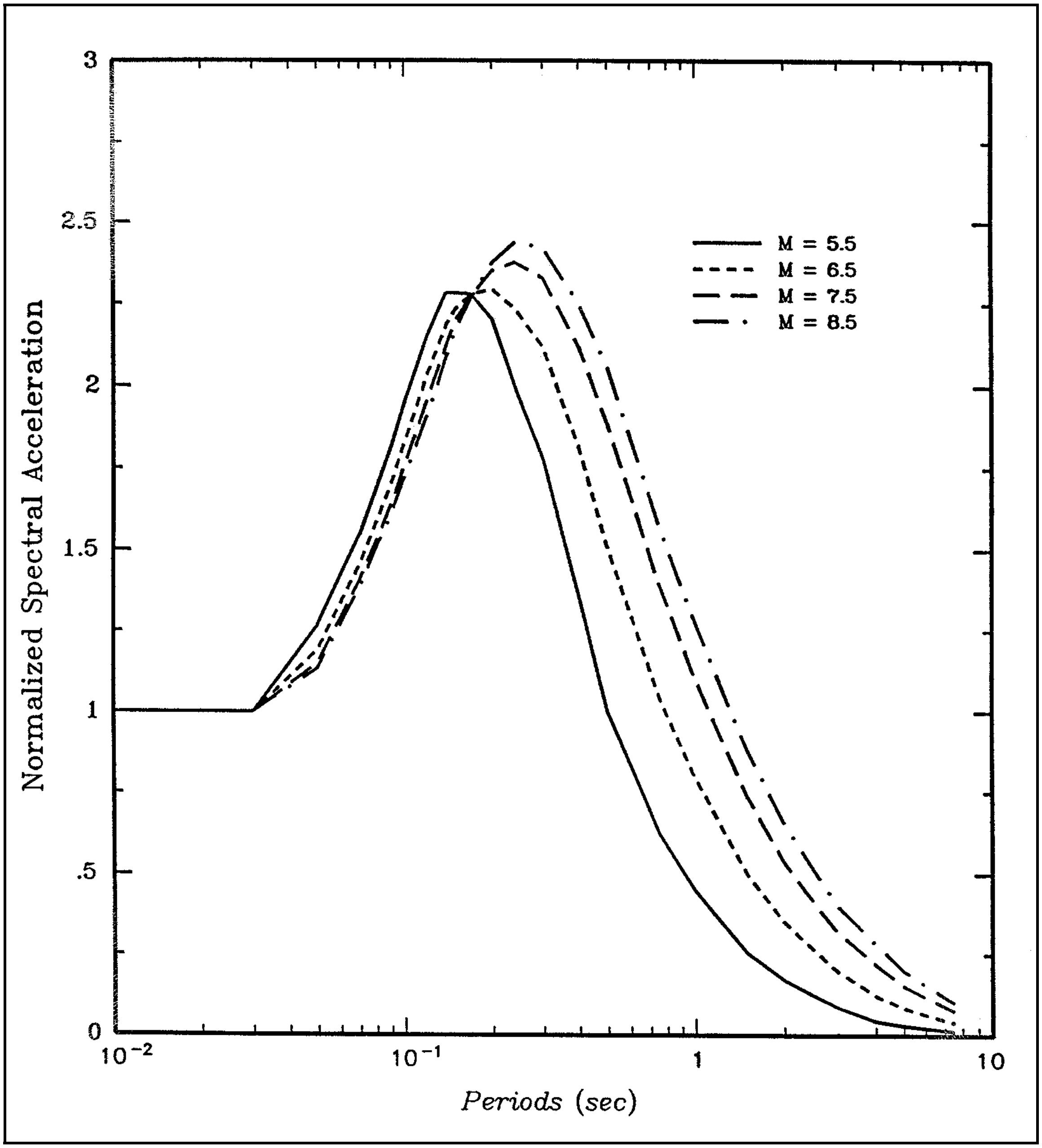


Figure 3-3. Effect of magnitude *M* on response spectral shape of rock motions based on attenuation relationships of Sadigh et al. (1993), 30-km distance from source to site, 5 percent damping

c. Effect of local subsurface conditions on ground motions.

(1) General. It is well established that local soil conditions have a major effect on the amplitude and response spectral characteristics of earthquake ground motions. It was demonstrated again by the dramatic differences in ground motions in different parts of Mexico City in the 1985 Mexico earthquake, and in different locations in the San Francisco Bay Area in the 1989 Loma Prieta earthquake.

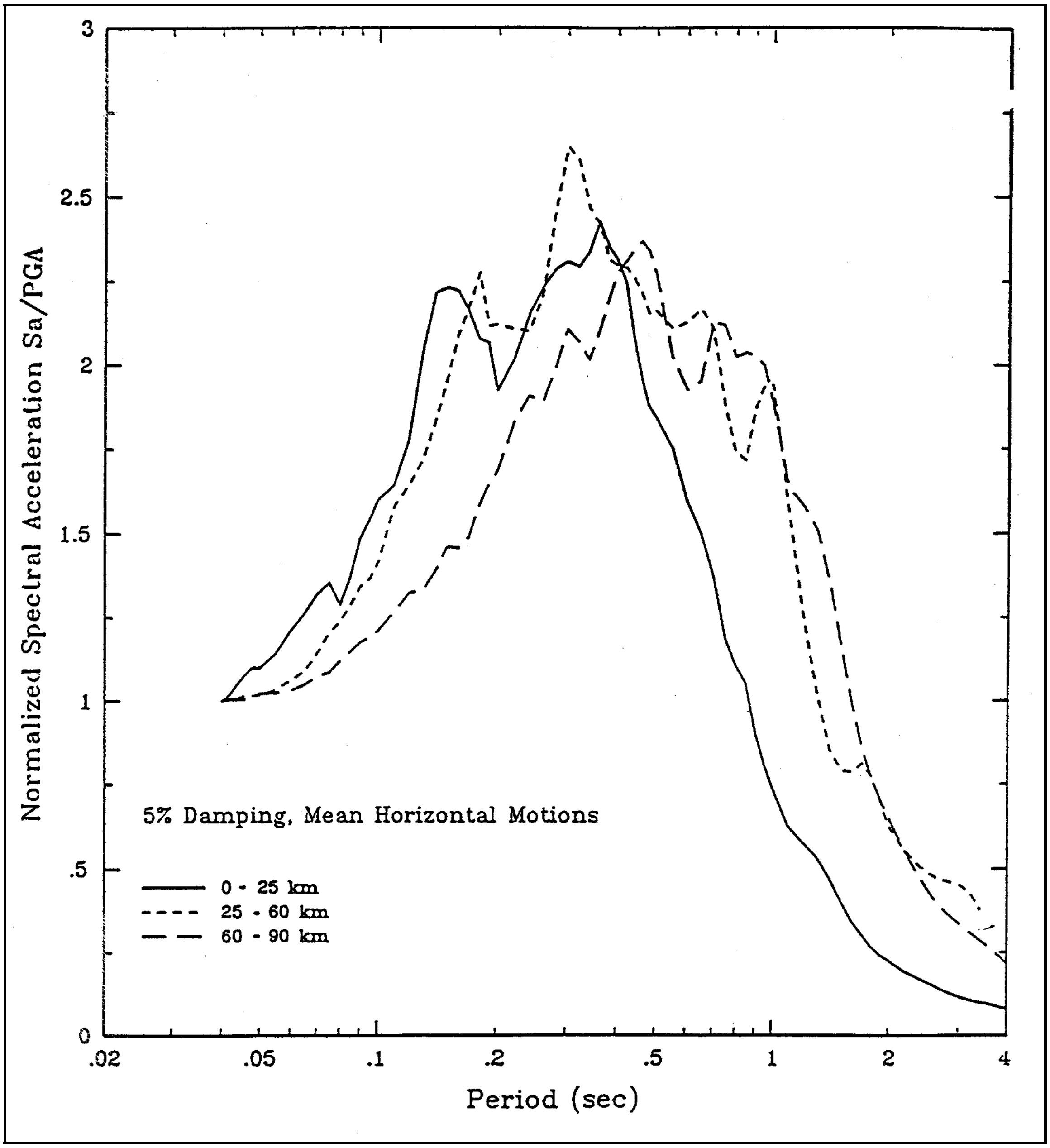


Figure 3-4. Variation of spectral shape with distance for rock recordings of the October 17, 1989, Loma Prieta earthquake

(2) Site amplification effects. Recordings obtained on different soil conditions and analytical studies indicate that soil amplification is dependent on the type and depth of soil. Figure 3-7 illustrates the amplification of response spectra of a soft soil site recording (Treasure Island site) relative to an adjacent rock site recording (Yerba Buena site) during the Loma Prieta earthquake. The effects illustrated in Figure 3-7 are qualitatively typical of those expected in soft soil for relatively low levels of ground motion (peak rock acceleration less than about 0.4 g). However, for higher levels of ground motion, higher soil damping due to nonlinear soil behavior tends to result in deamplification of high-frequency response spectra and peak ground accelerations a_{max} while longer period components continue to be amplified but

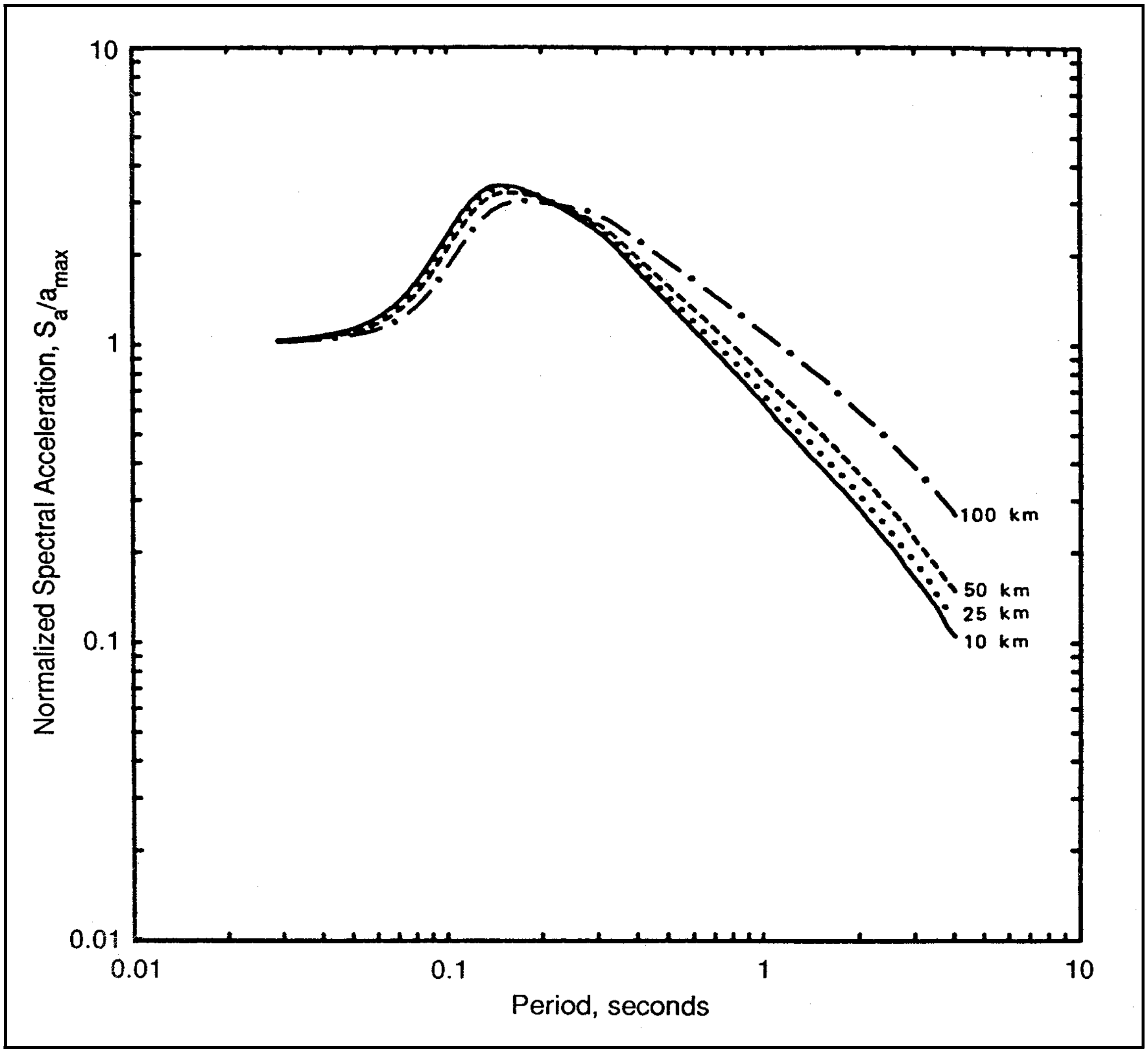


Figure 3-5. Effect of distance on response spectral shapes for a moment magnitude M_w 6.5 earthquake using western North American parameters (Silva and Green 1989, courtesy of Earthquake Engineering Research Institute, Oakland, CA)

by smaller amounts than for low levels of ground motion. Thus, site amplification effects are dependent on the level of ground motion as well as the soil characteristics. For peak ground acceleration, this dependence of amplification on ground motion level is illustrated by the relationship for soft soil developed by Idriss (1991a) shown in Figure 3-8. For response spectral values, the dependence of spectral amplifications on soil type and ground motion level are illustrated by the curves in Figure 3-9, which were the result of the National Center for Earthquake Engineering Research (NCEER)/Structural Engineers Association of California (SEAOC)/Building Seismic Safety Council (BSSC) workshop on site response held in 1992. The response spectral ratios shown in Figure 3-9 have been adopted into the NEHRP Provisions (BSSC 1994) and the Uniform Building Code (International Conference of Building Officials 1997).

(3) Effects on response spectral shape. The shape of the response spectrum is greatly influenced by the local subsurface conditions. This is illustrated in Figure 3-10 by the site-dependent spectral shapes developed by Seed, Ugas, and Lysmer (1976) for four different subsurface site classifications on the basis of statistical analysis of ground motion data. Although these spectral shapes could be updated

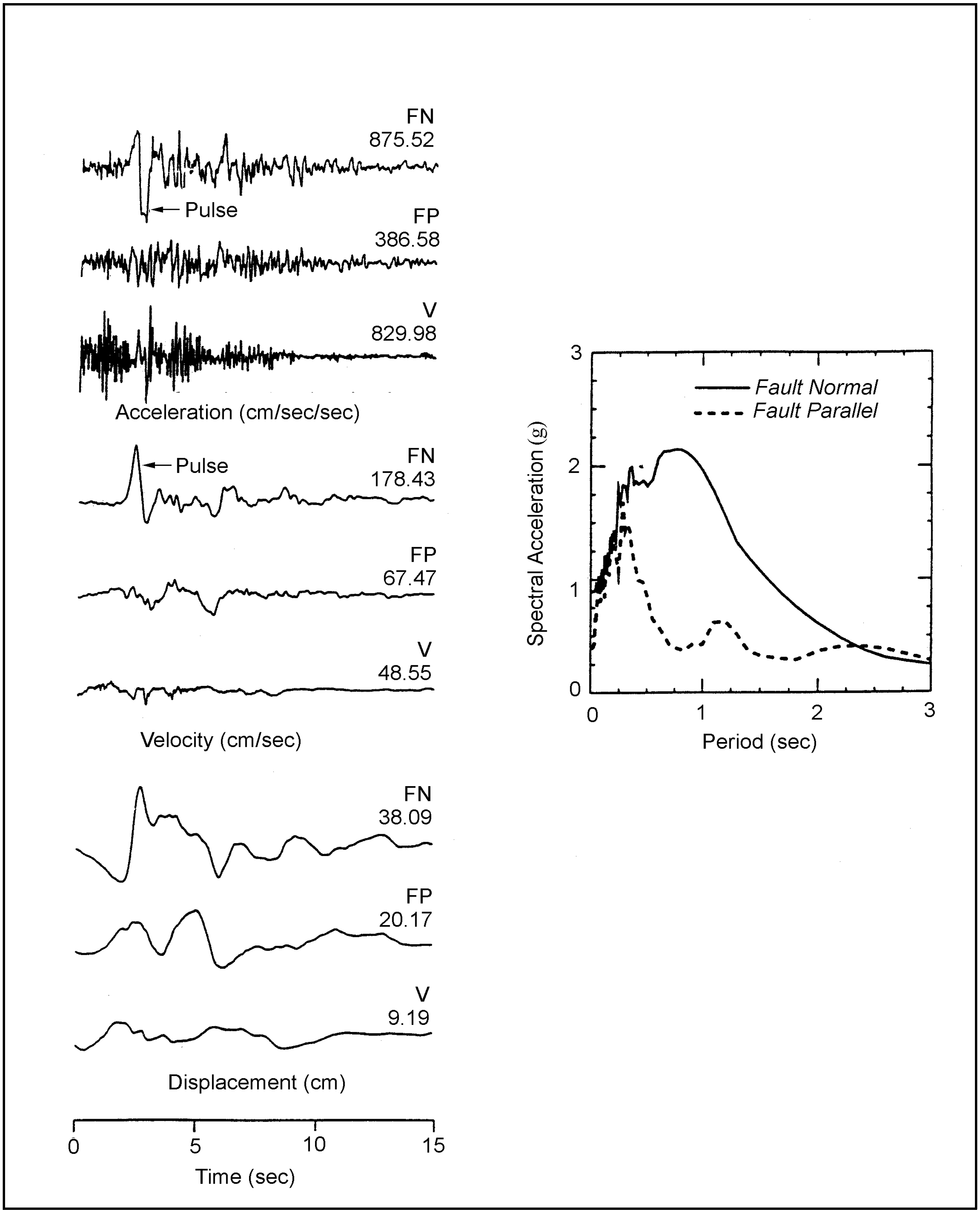


Figure 3-6. Time-histories and horizontal response spectra (5 percent damping) for the fault strike-normal (FN) and fault strike-parallel (FP) components of ground motion (V = vertical) for the Rinaldi recording obtained 7.5 km from the fault rupture during the 1994 Northridge, California, earthquake (Somerville (1997), courtesy of Multidisciplinary Center for Earthquake Engineering Research, State University of New York at Buffalo)

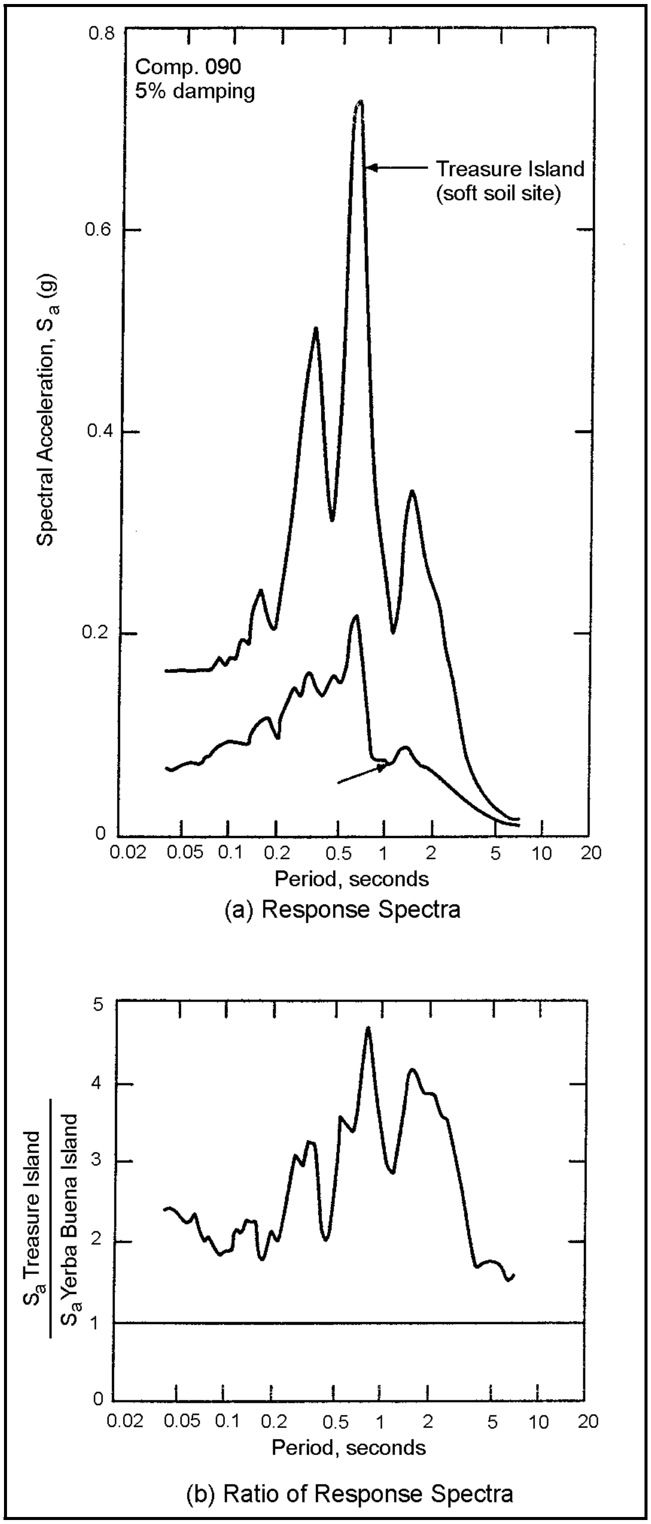


Figure 3-7. Response spectra and ratio of response spectra for ground motions recorded at a soft and nearby rock site during the 1989 Loma Prieta earthquake

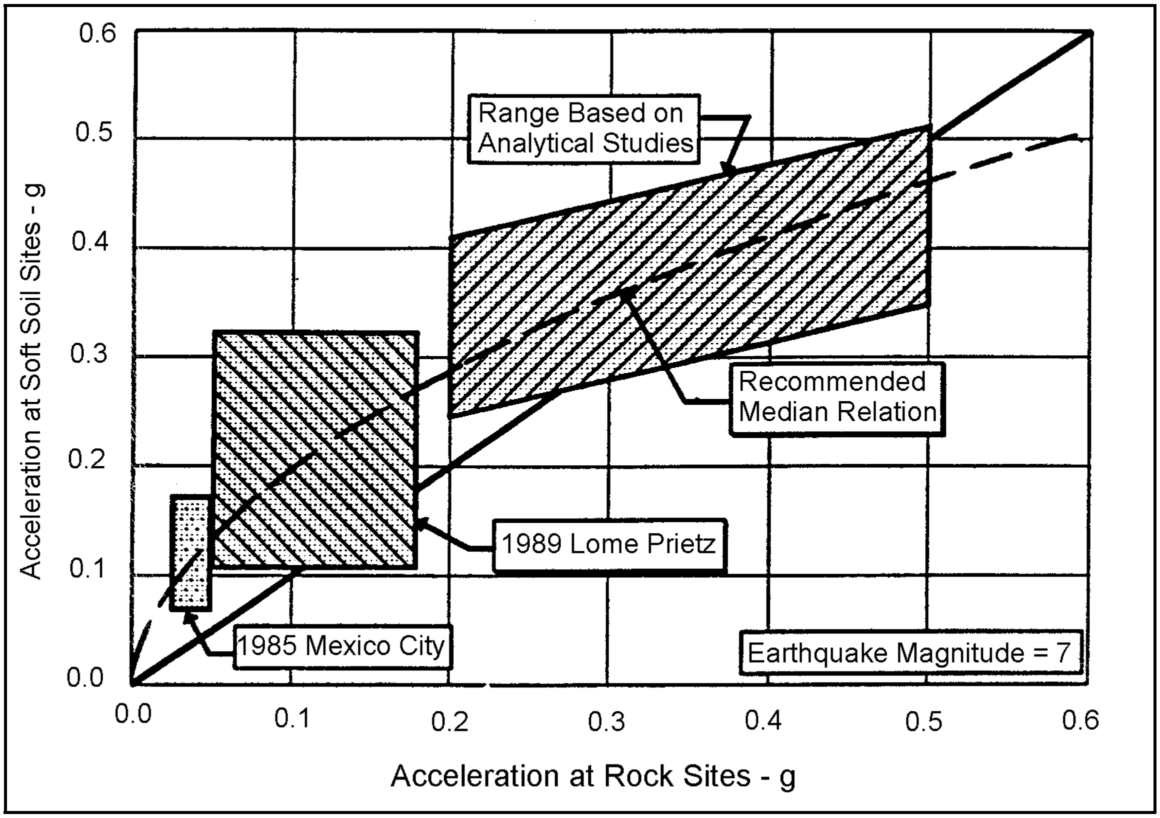
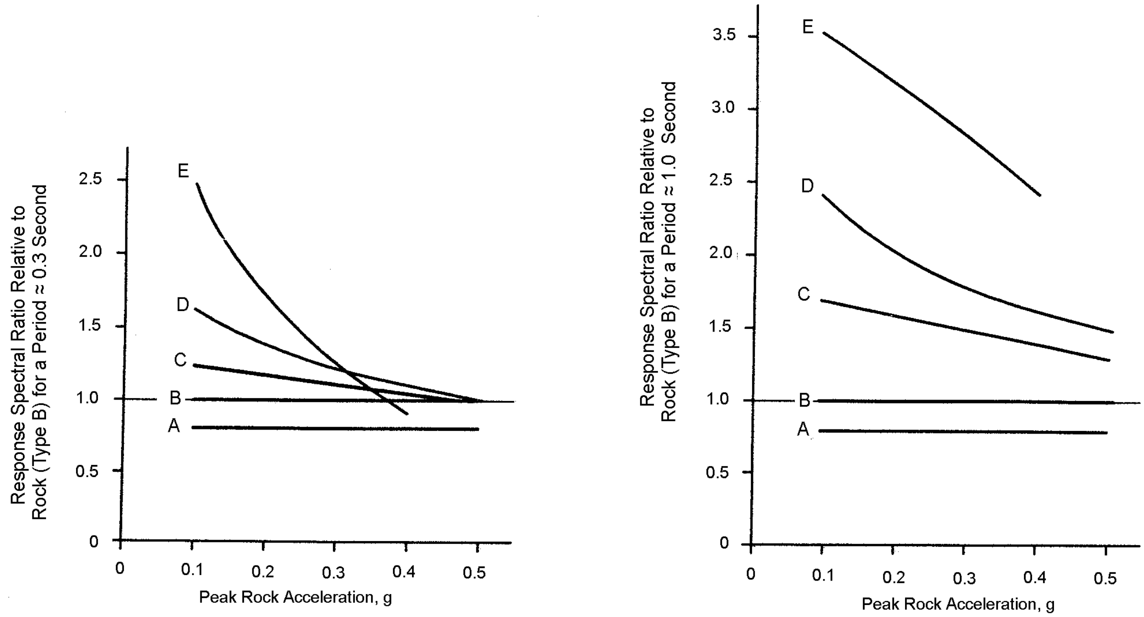


Figure 3-8. Variation of accelerations on soft soil sites versus rock sites (Idriss 1991a, courtesy of I. M. Idriss and Shamsher Prakesh, ed.)

using data from more recent earthquakes and specialized to different magnitudes, the spectral shapes developed by Seed, Ugas, and Lysmer (1976) have been widely used and have provided the basis for quantification of spectral shapes in building codes (Applied Technology Council 1978; Uniform Building Code (International Conference of Building Officials 1994).

3-4. Differences in Ground Motion Characteristics in Different Regions of the United States

a. Eastern versus western United States. Differences in strong ground motions between the WUS and EUS are due somewhat to earthquake source differences (somewhat higher stress drops in the EUS) but are currently believed to be due primarily to differences in travel path and site effects. In the stable intraplate region of the EUS, crustal rocks have higher shear wave velocities and lower damping than crustal rocks in the tectonically active interplate regions of the WUS. As a result, ground motions tend to attenuate more slowly with distance in the EUS than in the WUS. At the same time, the softer rocks within the upper 1 to 2 km of the crust in the WUS exhibit different site effects from those of the harder rocks of the EUS. Specifically, the WUS rocks, having higher damping and steeper velocity gradients with depth than EUS rocks, tend to damp out high-frequency components of motion while amplifying long-period components relative to EUS rocks. As a result of the interaction of these travel path and site effects, rock motions at relatively close source-to-site distances (within about 50 km) exhibit increased high-frequency motions (frequencies greater than about 5 to 10 Hz) but somewhat reduced long-period motions at EUS sites compared to WUS sites. Illustration of these differences in terms of response spectral shapes is shown by comparison of recorded ground motion data from California and Nahanni,



Site Classification	Description
A	Hard rock, shear wave velocity, $v_s >$ 1500 m/sec (5000 fps)
B	Rock, v_s 760 to 1500 m/sec (2500 to 5000 fps)
C	Very dense soil and soft rock, v_s 360 to 760 m/sec (1200 to 2500 fps)
D	Stiff soils, v_s 180 to 360 m/sec (600 to 1200 fps)
E	Soil profile with v_s <180 m/sec (600 ft/sec) or more than 3 m (10 ft) of soft clay with PI > 20, w ≥ 40%, and s_u <25 kPa (500 psf)

Note: v_s = weighted average shear wave velocity for the top 30 m (100 ft) of the soil profile

PI = plasticity index

w = water content

s_u = undrained shear strength

Figure 3-9. Response spectral ratios relative to rock for different site classifications and ground motion levels (BSSC 1994)

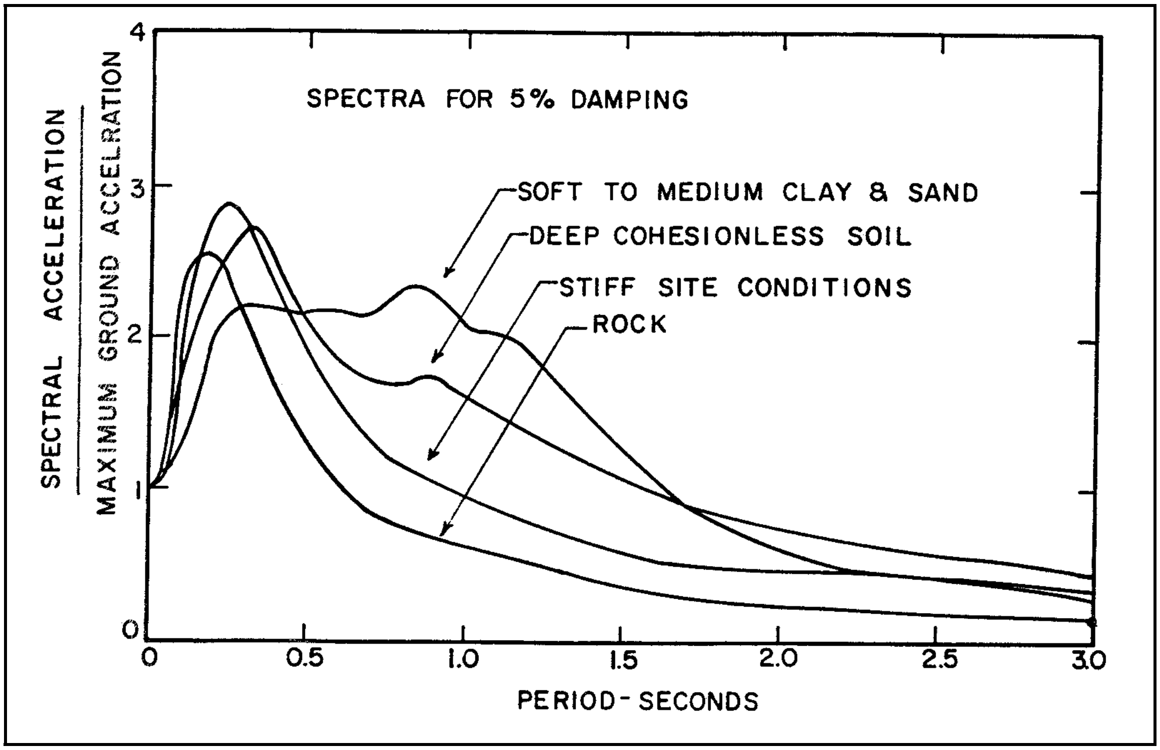


Figure 3-10. Average acceleration spectra for different site conditions (Seed, Ugas, and Lysmer 1976, courtesy of Seismological Society of America)

Canada, in Figure 3-11 (Nahanni is located in an EUS-like tectonic environment). In terms of absolute response spectra, these differences are illustrated in Figures 3-12 and 3-13, where response spectra for relatively close source-to-site distances have been calculated using the theoretical model of Silva and Green (1989). As distance increases, the effects of travel path attenuation begin to dominate over the local site effects, leading to higher ground motions in the EUS over a broader frequency range.

b. Subduction zone versus shallow crustal earthquakes. The collision of tectonic plates of the earth in subduction zones has caused numerous large and relatively deep earthquakes (e.g., in subduction zones in Japan; west coast of Central and South America; coastal northwest California, Oregon, and Washington; Alaska; Puerto Rico; and many other areas). Analyses of ground motion data from subduction zone earthquakes indicate that the main difference between ground motions from subduction zone earthquakes and ground motions from WUS shallow crustal earthquakes is a slower rate of attenuation for the subduction zone events. This is illustrated in Figure 3-14 in which attenuation of peak rock acceleration from shallow crustal WUS earthquakes is compared with that from subduction zone earthquakes as determined by Youngs et al. (1993a). Attenuation curves for two types of subduction zone earthquakes are shown in Figure 3-14—interface earthquakes occurring at the interface between the subducting tectonic plate and the overriding plate, and intraslab earthquakes occurring within the subducting plate. Analyses by Youngs, Day, and Stevens (1988) and Youngs et al. (1993a) also suggest that ground motions from subduction zone earthquakes have response spectral shapes that are lower in the long-period range than response spectral shapes for WUS shallow crustal earthquakes (Figure 3-15).

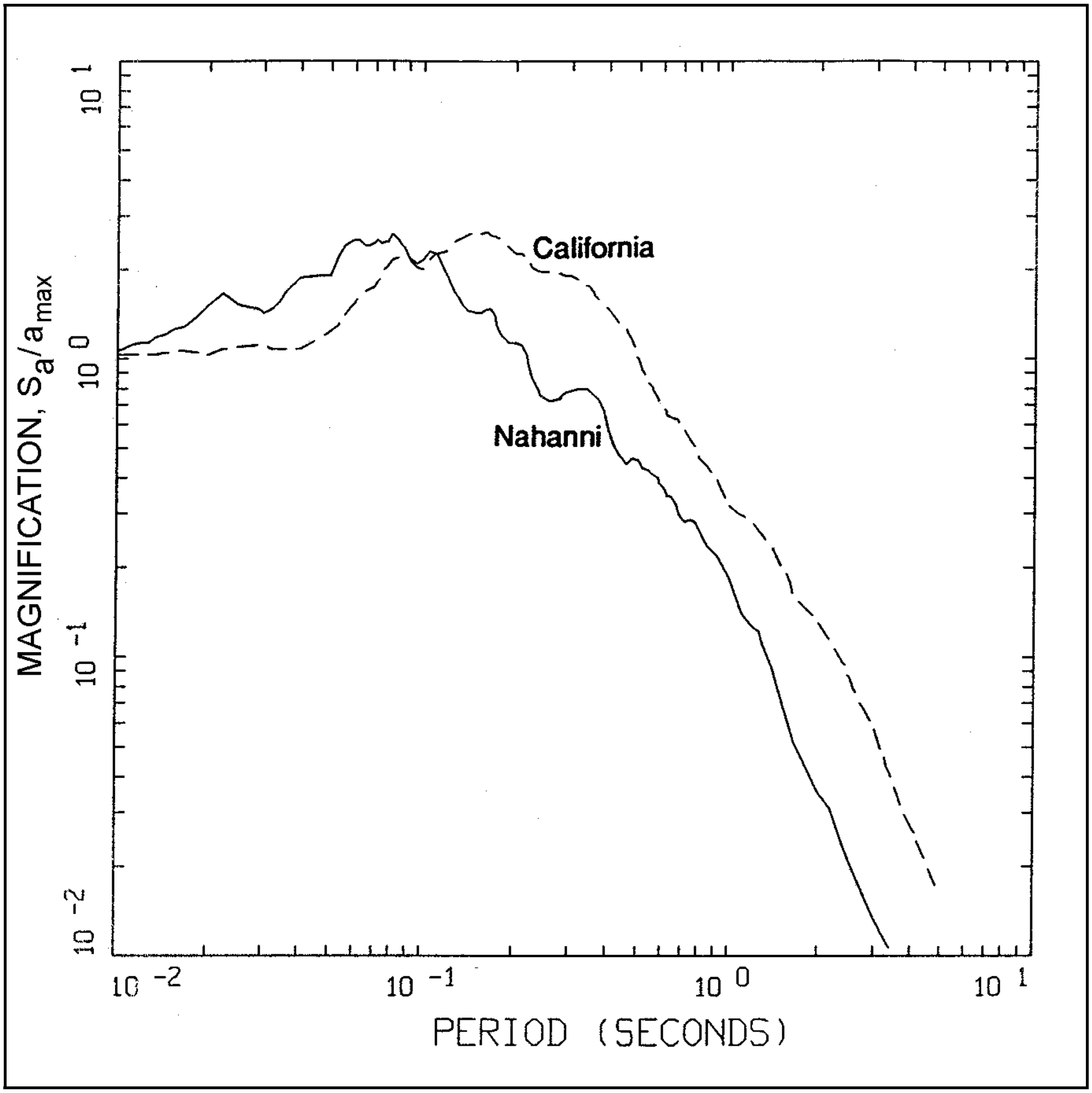


Figure 3-11. Comparison of average 5 percent damped response spectral shapes (S_a/a_{max}) computed from strong-motion data recorded at rock sites in Nahanni, Canada, and California for M_w 5.3 earthquakes (Darragh et al. 1989)

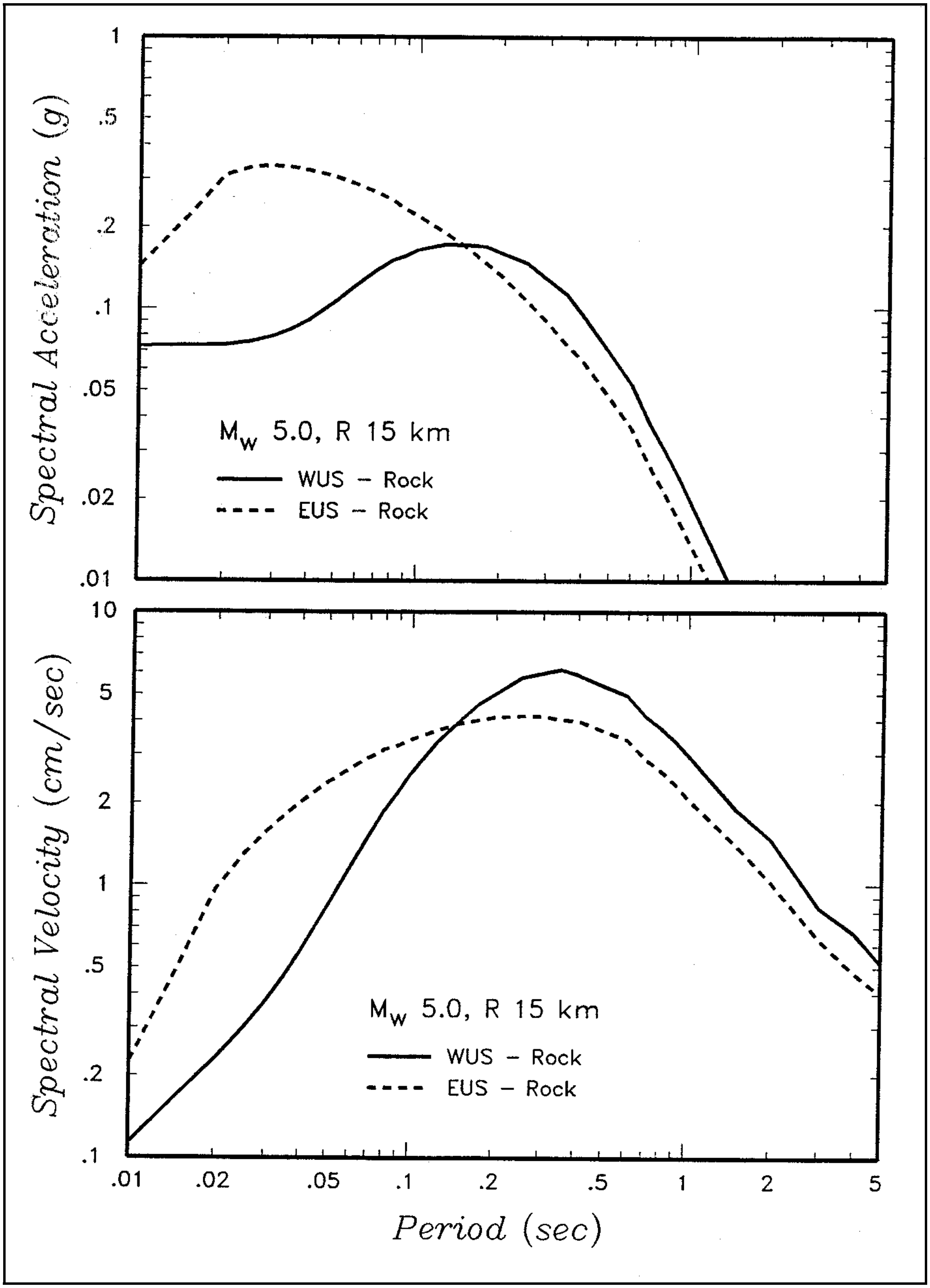


Figure 3-12. Comparison of response spectra for a magnitude 5 earthquake at 15 km using WUS and EUS attenuation relationships (calculated using Band-Limited-White-Noise/Random Vibration Theory (BLWN/RVT) model as formulated by Silva and Green 1989)

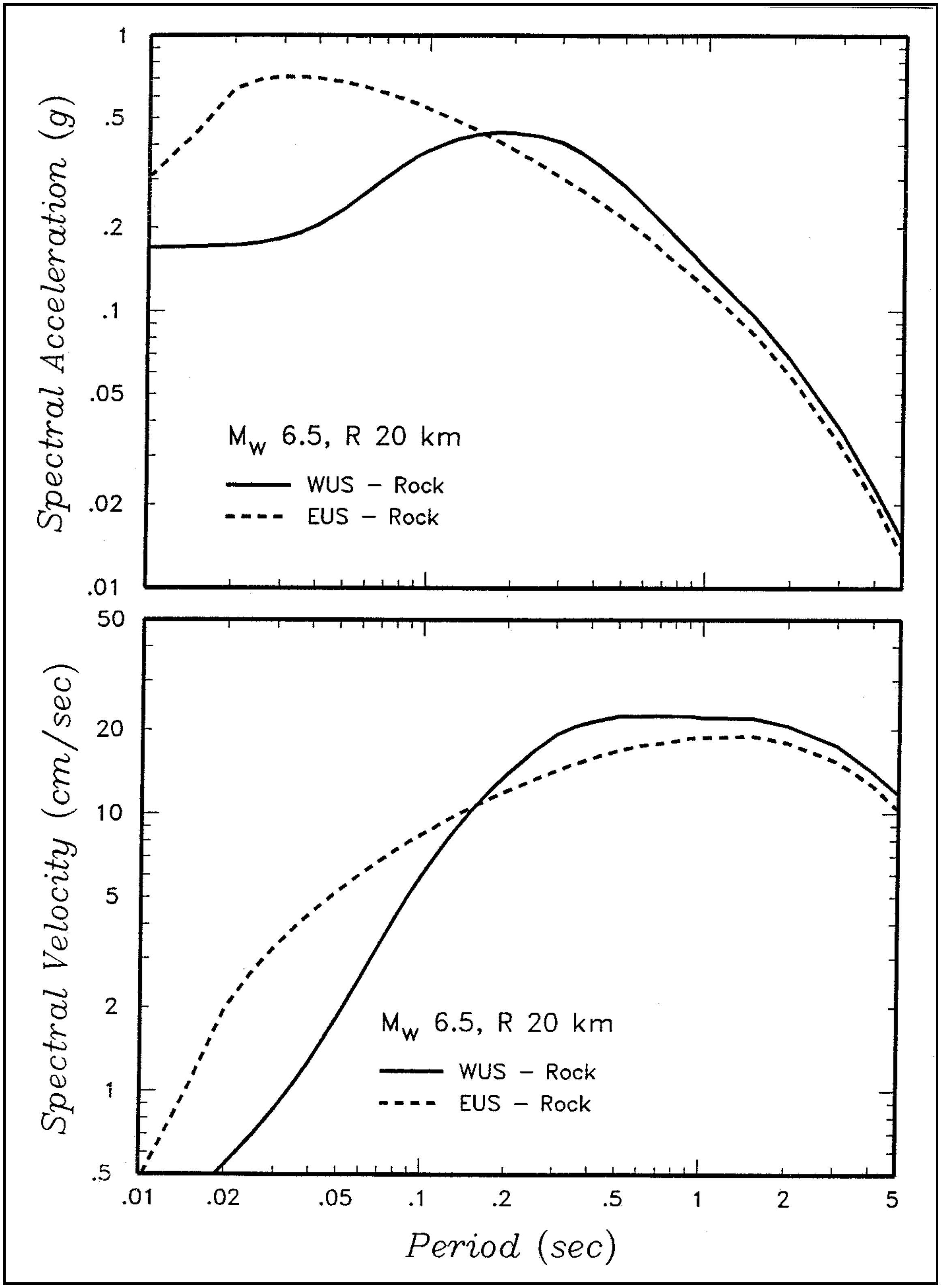


Figure 3-13. Comparison of response spectra for a magnitude 6.5 earthquake at 20 km using WUS and EUS attenuation relationships (calculated using BLWN/RVT model as formulated by Silva and Green 1989)

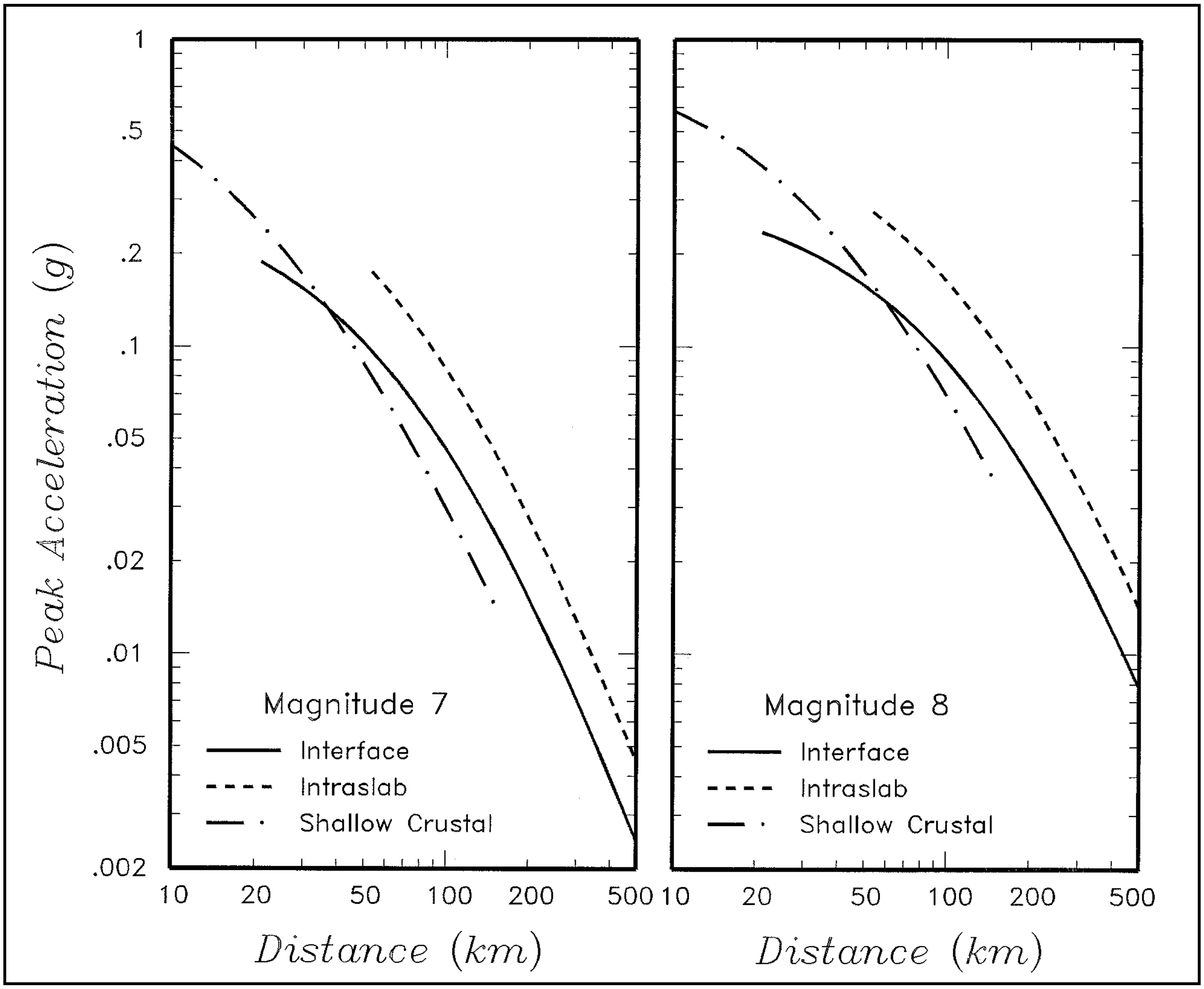


Figure 3-14. Comparison of median peak accelerations on rock from subduction zone earthquakes with peak accelerations from WUS shallow crustal earthquakes

Section II
Deterministic Procedures for Developing
Site-Specific Response Spectra

3-5. Summary of Alternative Procedures

Two basic approaches can be considered in developing design response spectra using a deterministic approach (deterministic seismic hazard analysis, or DSHA): Approach 1, anchoring response spectral shape to peak ground acceleration, and Approach 2, estimating the spectrum directly. These basic approaches are described below. This is followed in paragraphs 3-6 and 3-7 by an elaboration of the application of the two approaches to rock sites and soil sites, respectively.

a. Approach 1 - Anchoring response spectral shape to peak ground acceleration. Approach 1 is a three-part procedure in which peak ground acceleration is estimated, a response spectral shape is selected, and the shape is then multiplied by the peak ground acceleration to obtain the response spectrum, i.e.,

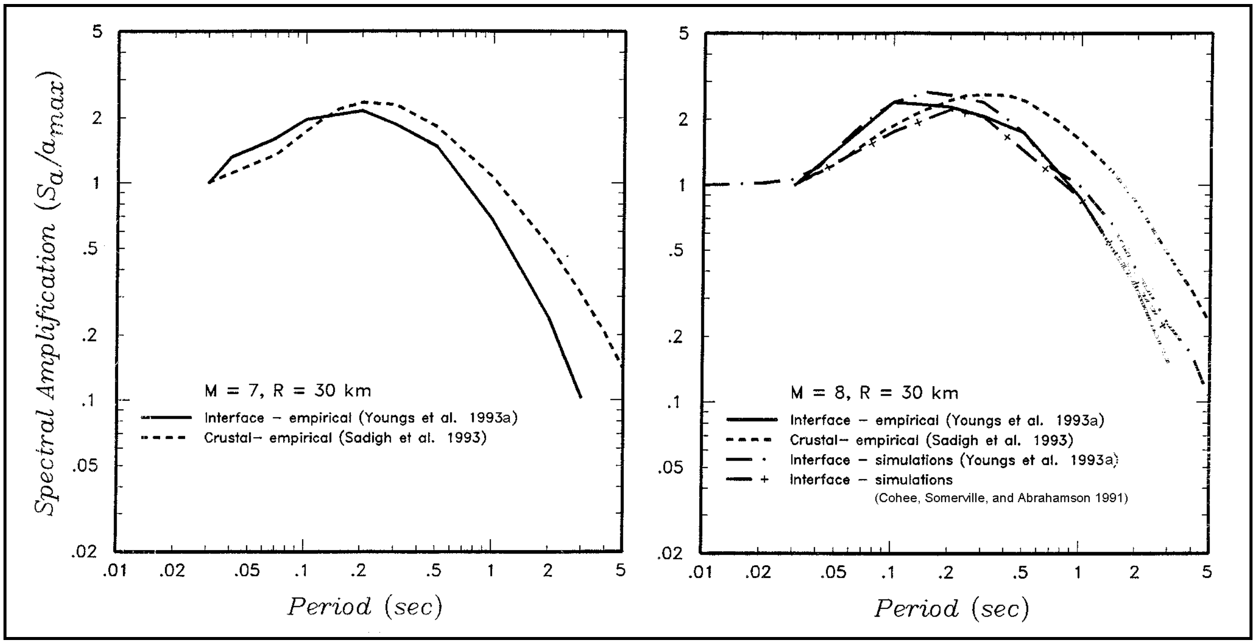


Figure 3-15. Comparison of spectral shapes using WUS and subduction zone attenuation relationships

(1) Step 1: Estimate peak ground acceleration (PGA).

(2) Step 2: Select response spectral shape, which is the curve of spectral amplification factors, (SA_T/PGA), where SA_T is spectral acceleration at period T.

(3) Step 3: Obtain response spectrum as the product of the peak ground acceleration and the spectral shape, SA_T = PGA × (SA_T/PGA). This approach is often referred to as "anchoring" the spectral shape to the peak ground acceleration. A variation on this approach is to estimate peak ground velocity (and, if desired, peak ground displacement) as well as peak ground acceleration and multiply these ground motion parameters by the appropriate spectral amplification factors (Newmark and Hall 1978, 1982); the Newmark and Hall procedure is summarized in Appendix B.

b. Approach 2 - estimating the spectrum directly. In Approach 2 the response spectrum ordinates are estimated directly as a single process. In general, there are three different approaches within Approach 2 for directly estimating response spectra: using response spectral attenuation relationships; performing statistical analysis of spectra from selected ground motion records; and theoretical (numerical) modeling. These approaches are briefly outlined below.

(1) Using response spectral attenuation relationships. Attenuation relationships have been developed by several investigators for response spectral values of ground motions (spectral acceleration or spectral pseudo-relative velocity) at selected periods of vibration by performing statistical regression analyses of ground motion data and by conducting theoretical analyses. Relationships have been developed for different site conditions and tectonic environments. Specific relationships are presented in paragraph 3-6 for rock sites and paragraph 3-7 for soil sites. These relationships can be used to make period-by-period estimates of response spectral values, given the design earthquake magnitude and distance. (The zero-period spectral value is obtained from the corresponding attenuation relationship for the peak ground acceleration, i.e., zero-period spectral acceleration (ZPA) = PGA.)

Typically, these relationships have been developed for 5 percent damping; and ratios between spectral values at different damping ratios (e.g., Newmark and Hall 1978, 1982; Appendix B) are used to obtain the corresponding spectra for other damping ratios.

(2) Performing statistical analysis of ground motion data. Spectral attenuation relationships discussed above are based on all the available applicable ground motion data, and they typically cover a wide range of earthquake magnitudes and distances. However, for a specific magnitude and distance, it may be possible to obtain an improved or a comparative estimate of the response spectrum by performing statistical analysis using a set of response spectra of ground motion records from earthquakes having magnitudes and distances that are close to the design magnitude and distance. The data set is typically a subset of the data used to develop attenuation relationships. The analysis may consist of direct, period-by-period statistical analysis of the data. However, it is also possible to "scale" or adjust each response spectral value of each record to values for the design magnitude and distance and then do statistical analysis of the scaled data set. The attenuation relationships for response spectral values (discussed in (1) above) are used to perform the scaling. This approach of scaling before performing statistical analyses is recommended unless the magnitudes and distances for the data set are closely bunched around the design magnitude and distance. Appendix C illustrates the approach of statistical analyses of a set of scaled response spectra. A particular type of statistical analysis that has been used for many nuclear power plant sites in the EUS and for other sites and locations as well is termed a "random earthquake" analysis. This analysis is performed to estimate the response spectrum at a site due to a randomly located ("floating") earthquake within the vicinity of the site, i.e., when its location cannot be assigned to a specific geologic structure at a specific distance from the site. After the design magnitude of the random earthquake is selected, a statistical analysis is made of response spectra of ground motion records from earthquakes having magnitudes close to the design magnitude, recorded on site conditions similar to those for the actual site, and recorded within a selected source-to-site distance, typically 25 km. A random earthquake analysis can also be performed using attenuation relationships. Appendix D illustrates a random earthquake analysis.

(3) Performing theoretical (numerical) ground motion modeling. The state of the art for theoretical (numerical) modeling of ground motions is being vigorously advanced and is being increasingly used for site-specific project applications. Various methods attempt to simulate the earthquake rupture, the propagation of seismic waves from earthquake source to site, and/or the effect of local site conditions. A number of methods have been developed. These methods warrant consideration as a supplementary means for ground motion estimation. They can be particularly useful for extrapolating to conditions that lie outside those represented by the database of strong motion recordings. The methods vary considerably in degree of complexity and sophistication. One relatively simple model that has been used increasingly to simulate earthquake rupture and source-to-site wave propagation is the Band Limited White Noise/Random Vibration Theory (BLWN/RVT) Model (Atkinson 1984; Atkinson and Boore 1995; Boore 1983, 1986; Boore and Atkinson 1987; Boore and Joyner 1991; Hanks and McGuire 1981; McGuire, Toro, and Silva 1988; Silva and Green 1989). This method has been applied particularly in the EUS because of the relative scarcity of ground motion data in the EUS, and attenuation relationships for the EUS have been developed using this model. A particular form of theoretical analysis applicable to soil sites is "site response analysis," or "ground response analysis" in which the objective is to assess the modifying influence of the local soil conditions on rock motions estimated for the site and, in this manner, estimate the motions at the ground surface of the soil site. Site response analyses are discussed in paragraph 3-7 in the context of their use in estimating response spectra for soil sites.

c. Relative advantages of Approach 1 and Approach 2. Approach 2, estimating the spectrum directly, should be used rather than Approach 1, anchoring spectral shape to peak ground acceleration, because, as was outlined in Section I, spectral shapes depend on more than just the site conditions (i.e., on tectonic environment, earthquake magnitude, and other factors), yet the readily available and widely used spectral shapes generally incorporate only the effect of the local site conditions. Currently available procedures, relationships, and data enable response spectra to be estimated as a single process in Approach 2. When spectra are estimated using Approach 2, it is often useful to make comparative estimates using Approach 1. In paragraph 3-6 procedures and relationships for Approaches 1 and 2 for developing response spectra for rock sites are discussed. A similar presentation is made in paragraph 3-7 for soil sites.

3-6. Developing Site-Specific Spectra for Rock Sites

a. Using Approach 1 - Anchoring rock response spectral shape to peak rock acceleration.

(1) Estimating peak rock acceleration. Recently developed attenuation relationships for estimating peak rock acceleration for shallow crustal earthquakes in the WUS, for the EUS, and for subduction zones are summarized in Table 3-1. The relationships for the WUS shallow crustal earthquakes are better constrained than those for the other tectonic environments because of the relative abundance of strong motion data for WUS shallow earthquakes. The peak acceleration attenuation relationships in Table 3-1 generally include the standard deviations of the estimates, from which 84th percentile values may be obtained.

Table 3-1
Summary of Recently Developed Attenuation Relationships for Peak Ground Acceleration

Tectonic Environment	Relationship	Site Condition
WUS shallow earthquakes	Idriss (1991b)[1]	Rock
	Idriss (1991a)[1]	Soft soil
	Sadigh et al. (1993)[1,2]	Rock
	Abrahamson and Silva (1997)[1,2]	Rock and deep firm soil
	Campbell (1997)[1,2]	Alluvium, soft rock, hard rock
	Boore, Joyner, and Fumal (1997)[1]	Four site classifications based on shear wave velocity
	Sadigh et al. (1997) [1]	Rock and deep firm soil
EUS	Boore and Joyner (1991)	Deep soil
	Frankel et al. (1996)	Hard rock
	Atkinson and Boore (1997)	Hard rock
	Toro, Abrahamson, and Schneider (1997)	Hard rock
Subduction zone	Crouse (1991)	Firm soil
	Molas and Yamazaki (1995)	Rock; hard soil; medium soil; soft soil
	Youngs et al. (1997)	Rock and deep soil
Subduction zone and shallow earthquakes (no distinction)	Fukushima and Tanaka (1990)	Rock; hard soil; medium soil; soft soil
	Krinitzsky, Chang, and Nuttli (1987)	Hard site and soft site

[1] Includes a factor for type of faulting.
[2] Including vertical ground motions.

(2) Estimating response spectral shape and the response spectrum. Available spectral shapes for rock site conditions are summarized in Table 3-2. Multiplying spectral shape times peak ground acceleration results in the absolute response spectrum. Using the Newmark and Hall (1978, 1982) approach (Appendix B), peak ground acceleration is multiplied by the acceleration amplification factor, peak ground velocity by the velocity amplification factor, and peak ground displacement by the displacement amplification factor. Newmark and Hall's recommended relationships between peak ground acceleration, peak ground velocity, and peak ground displacement for rock site conditions may be used to estimate peak ground velocity and peak ground displacement, given peak ground acceleration; or peak ground velocity and peak ground displacement may be independently estimated. Note that the relationships in Table 3-2 of Seed, Ugas, and Lysmer (1976), Mohraz (1976), Applied Technology Council (1978), and Newmark and Hall (1978, 1982), do not explicitly incorporate the important effects of magnitude on spectral shape. The relationships of Mohraz (1976) and Newmark and Hall (1978, 1982) can indirectly incorporate the effects of magnitude through its effect on peak ground velocity and peak ground displacement if these parameters are estimated independently from peak ground acceleration (Appendix B).

Table 3-2
Summary of Available Response Spectral Shapes of Ground Motions

Tectonic Environment	Relationship	Site Conditions
WUS shallow earthquakes	Seed, Ugas, and Lysmer (1976)	Rock; stiff soil; deep cohesionless soil; soft to medium clay and sand
	Mohraz (1976)	Rock; shallow alluvium; moderately deep alluvium; alluvium
	Applied Technology Council (1978)[1]	Rock or shallow stiff soil; deep stiff soil; soft soil
	Newmark and Hall (1978, 1982)	Rock; firm soil
	Sadigh et al. (1997)	Rock and deep firm soil
WUS shallow earthquakes; EUS	Silva and Green (1989)	Rock
Subduction zones	Youngs et al. (1997)	Rock and deep soil

[1] These spectral shapes are based primarily on Seed, Ugas, and Lysmer (1976). The Applied Technology Council (1978) spectral shapes are also the spectral shapes that appear in the 1994 Uniform Building Code (International Conference of Building Code Officials 1994) and the 1995 Recommended Lateral Force Requirements and Commentary ("Blue Book") of the Seismology Committee of the Structural Engineers Association of California (SEAOC 1996).

(3) Uncertainty in response spectral prediction. Response spectra obtained using the median or mean peak ground acceleration attenuation relationships and the median or mean spectral shapes summarized above result in median (50th percentile) or mean ground motion estimates, given a design earthquake magnitude and distance. Additional consideration must be given if a higher percentile ground motion is to be predicted, for example the 84th percentile (median plus standard deviation) ground motion. Spectra for the 84th percentile can be estimated by multiplying median or mean peak ground acceleration by 84th percentile spectral shapes. For example, Seed, Ugas and Lysmer (1976) present 84th percentile shapes as well as mean shapes. However, this procedure results in a lesser degree of conservatism in the very short period part of the spectrum because the anchor value of peak ground acceleration (equal to the zero-period spectral acceleration, ZPA) is at the median or mean level. In order to have a uniform degree of conservatism throughout the period range, the peak ground acceleration and the very short period part of the spectrum should be adjusted upward to the 84th percentile level. Alternatively, the entire median response spectrum may be scaled upward by a factor to the approximate 84th percentile level. When the entire spectrum is scaled, often the standard deviation in peak ground acceleration is used to obtain a scaling factor. This is an approximation, since the standard deviation has

been found to vary with period of vibration. (The period dependence can be directly accounted for when using Approach 2.) The 84th percentile amplification factors of the Newmark and Hall (1978, 1982) approach can be applied to median estimates of peak ground acceleration, velocity, and displacement to obtain 84th percentile spectral values (Appendix B, Table B-1). Again, a separate adjustment should be applied to raise the peak ground acceleration and very short period part of the spectrum to the 84th percentile level.

(4) Estimating vertical ground motion response spectra. Ratios of vertical to horizontal response spectral amplitudes can be used to estimate vertical response spectra, given an estimate of horizontal response spectra. Recent studies (e.g., Silva 1997) indicate that vertical-to-horizontal response spectral ratios are a function of period of vibration, earthquake source to site distance, earthquake magnitude, tectonic environment (WUS and EUS), and subsurface conditions (soil and rock). Figure 3-16 provides a guideline for ratios of vertical to horizontal spectral values on rock sites that is generally conservative for earthquake magnitudes equal to or less than about 6.5. However, if a facility is sensitive to short-period (less than 0.3 sec) vertical motions, and is located within 10 km of the earthquake source or the magnitude exceeds 6.5, further evaluation of vertical response spectra on rock is recommended because vertical response spectra can significantly exceed horizontal response spectra for these conditions.

b. Using Approach 2 - Estimating the rock response spectrum directly.

(1) General. The three approaches discussed in paragraph 3-5*b* can be used: using response spectral attenuation relationships, performing statistical analyses of ground motion response spectra, and theoretical (numerical) modeling. The following paragraphs refer to particular relationships or methods.

(2) Using response spectral attenuation relationships. Recently developed attenuation relationships that can be used to predict horizontal rock response spectral values for WUS shallow crustal earthquakes, for EUS earthquakes, and for subduction zone earthquakes are summarized in Table 3-3. The spectral acceleration attenuation relationships summarized in Table 3-3 generally include the standard deviations of the estimates, from which 84th percentile values may be obtained. As illustrated in Figures 3-12 and 3-13, EUS attenuation models characteristically estimate much higher spectral response in the short-period range (less than 0.1 to 0.2 sec) than estimated by relationships for the WUS. As discussed in paragraph 3-4*a* the higher short-period response is attributed to the hardness of the rock in the EUS. Such short-period (high-frequency) motions may or may not be significant to the response and performance of hydraulic structures. The assessment of the significance of such motions should be made by the principal design engineer in collaboration with the seismic structural analyst and the materials engineer.

(3) Using statistical analyses of response spectra. Abundant strong motion data are available to permit statistical analyses of data sets for many design earthquake scenarios for WUS shallow crustal earthquakes. As noted in paragraph 3-5*b*(2), statistical "random earthquake" analyses have been made at many EUS sites because, in general, discrete faults have not been identified in the EUS. Yet it was desired to model the possibility of an earthquake (usually of moderate magnitude, in the range of magnitude 5 to 6) occurring near the site. Many of these analyses have been carried out using WUS ground motion data because only a few records from moderate magnitude earthquakes are available in the EUS. Such analyses may be reasonable for estimating longer period rock motions but would apparently underestimate shorter period (less than 0.1 to 0.3 sec) ground motions at hard rock sites. In this case, an adjustment should be made for the short-period response spectral values, if these short-period motions are of significance to the structure under consideration. The adjustment can be made by comparing

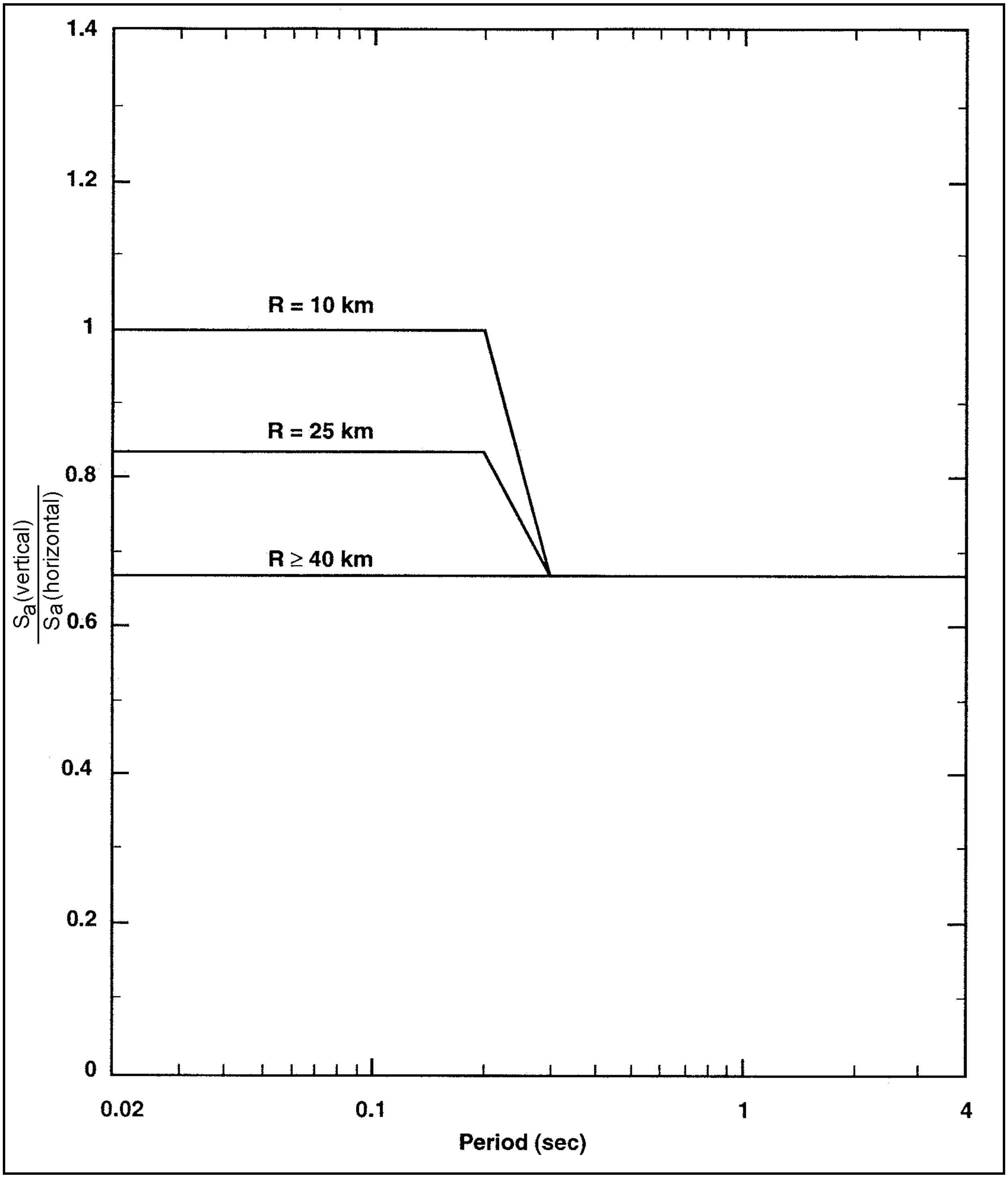


Figure 3-16. Simplified relationships between vertical and horizontal response spectra as a function of distance *R*

Table 3-3
Summary of Recently Developed Attenuation Relationships for Response Spectral Values of Ground Motions

Tectonic Environment	Relationship	Site Conditions
WUS shallow earthquakes	Idriss (1991b)[1]	Rock
	Sadigh et al. (1993)[1,2]	Rock
	Abrahamson and Silva (1997)[1,2]	Rock and deep firm soil
	Campbell (1997)[1,2]	Alluvium, soft rock, hard rock
	Boore, Joyner, and Fumal (1997)[1]	Four site classifications based on shear wave velocity
	Sadigh et al. (1997)[1]	Rock and deep firm soil
EUS	Boore and Joyner (1991)	Deep soil
	Frankel et al. (1996)	Hard rock
	Atkinson and Boore (1997)	Hard rock
	Toro, Abrahamson, and Schneider (1997)	Hard rock
Subduction zone	Crouse (1991)	Firm soil
	Youngs et al. (1997)	Rock and deep soil

[1] Includes a factor for type of faulting.
[2] Including vertical ground motions.

response spectrum amplitudes predicted by EUS and WUS attenuation relationships for rock (Table 3-3). As illustrated in Appendix D, a random earthquake analysis can also be carried out using attenuation relationships. Thus this analysis can be performed using the EUS spectral attenuation relationships that predict higher short-period motions at hard rock sites.

(4) Using theoretical (numerical) modeling techniques. The techniques discussed in paragraph 3-5*b*(3) can be used. These techniques attempt to simulate the earthquake rupture and the propagation of seismic waves from the earthquake source to the site.

(5) Estimating vertical ground motion response spectra. The available recently developed attenuation relationships for response spectral values of vertical rock motions are summarized in Table 3-3. These relationships can be used directly to estimate vertical rock response spectra. The vertical to horizontal spectral ratios discussed in paragraph 3-6*a*(4) and shown in Figure 3-16 can be used as a guide in estimating vertical response spectra, given an estimate of the horizontal spectra.

c. Developing acceleration time-histories of rock motions consistent with the design response spectrum.

(1) General. When acceleration time-histories of ground motions are required for the dynamic analysis of a structure, they should be developed to be consistent with the design response spectrum over the period range of significance for the structure, as well as have an appropriate strong motion duration for the particular design earthquake. Two general approaches to developing acceleration time-histories are selecting a suite of recorded motions and synthetically developing or modifying one or more motions.

These approaches are discussed below. For either approach, when near-source earthquake ground motions are modeled, it may be desirable that an acceleration time-history include a strong intermediate- to long-period pulse to model this particular characteristic of ground motion often observed in the near field (paragraph 3-3*b*(4) and Figure 3-6).

(2) Selecting recorded motions. Every earthquake produces a unique set of acceleration time-histories having characteristics that depend on the earthquake magnitude and other source characteristics, distance, attenuation and other travel path characteristics, and local site conditions. The response spectrum of any individual ground motion accelerogram has peaks and valleys that occur at different periods of vibration. Thus, the response spectrum of any single accelerogram is unlikely to match the developed smooth design response spectrum. Typically, when recorded motions are selected, it is necessary to choose a suite of time-histories (typically at least four) such that, in aggregate, valleys of individual spectra that fall below the design curve are covered by peaks of other spectra and (preferably) the exceedance of the design spectrum by individual spectral peaks is not excessive. For nonlinear analyses, it may be desirable to have additional time-histories because of the importance of pulse sequencing to nonlinear response (see also comments in (3) below). In the approach of selecting recorded motions, simple scaling of individual accelerograms by a constant factor is done to improve the spectral fit, but the wave form and the relative spectral content of the accelerograms are not modified. The advantage of selecting recorded motions is that each accelerogram is an actual recording, and thus the structure is analyzed for natural motions that are presumably most representative of what the structure could experience. The approach has the following disadvantages: multiple dynamic analyses are needed for the suite of accelerograms selected; although a suite of accelerograms is selected, there will typically be substantial exceedances of the smooth design spectrum by individual spectrum peaks; and although a reasonably good spectral fit may be achieved for one horizontal component, when the same simple scaling factors are applied to the other horizontal components and the vertical components for the records selected, the spectral fit is usually not as good for the other components.

(3) Synthetically developing or modifying motions. A number of techniques and computer programs have been developed either to completely synthesize an accelerogram or modify a recorded accelerogram so that the response spectrum of the resultant accelerogram closely fits or matches the design spectrum. It is preferred to use techniques that modify a recorded accelerogram rather than completely synthesize a motion since the recorded motion will have time-domain characteristics representative of actual ground motions. Two techniques that have been found to do a good job of spectrally modifying recorded motions are the frequency-domain RASCAL computer code developed for the Corps of Engineers by Silva and Lee (1987) and the time-domain technique developed by Lilhanand and Tseng (1988). These techniques preserve the basic time-domain character of the accelerogram yet provide an excellent match to a smooth spectrum. An example of the spectral matching technique is given in Figures 3-17 and 3-18. The RASCAL computer code was used in this case. Figure 3-17 illustrates the initial (recorded) acceleration, velocity, and displacement time-histories and the time-histories after the spectral matching process. Figure 3-18 illustrates the initial (recorded) acceleration response spectra, the smooth design response spectrum, and the response spectrum of the acceleration time-history after the spectral matching process. Figure 3-18 illustrates the very close spectral match that was achieved, while Figure 3-17 illustrates that the modified time-histories preserve the basic time-domain character of the original record. Synthetic techniques for developing time-histories have the following advantages: a good fit to the design spectrum can be achieved with a single accelerogram; the natural appearance and strong motion duration can be maintained in the accelerograms; and three component motions (two horizontal and one vertical) each providing a good spectral match can be developed, and these can be made statistically independent if desired; and the process is relatively efficient.

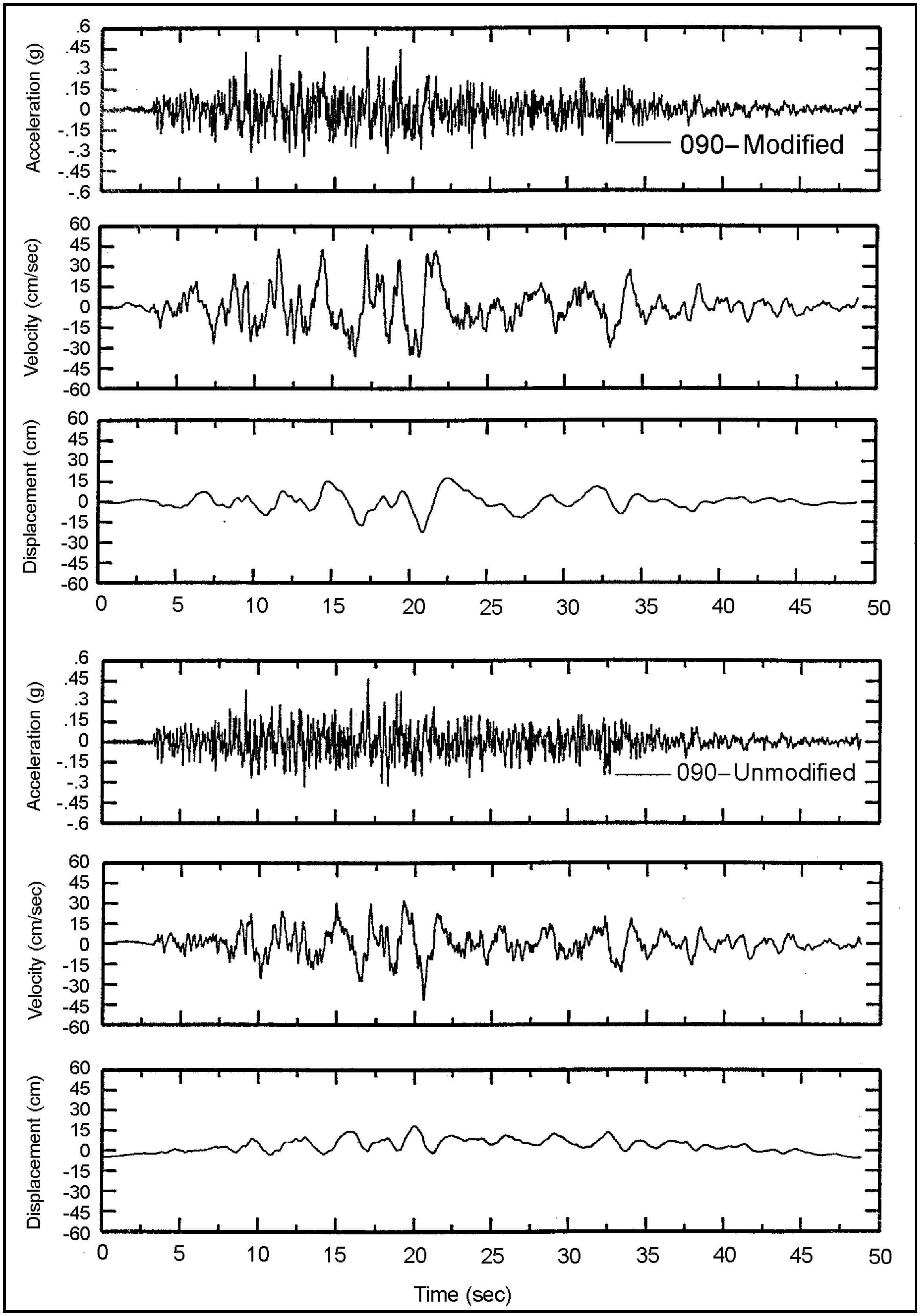


Figure 3-17. Example of original time-histories and time-histories after a spectral matching process

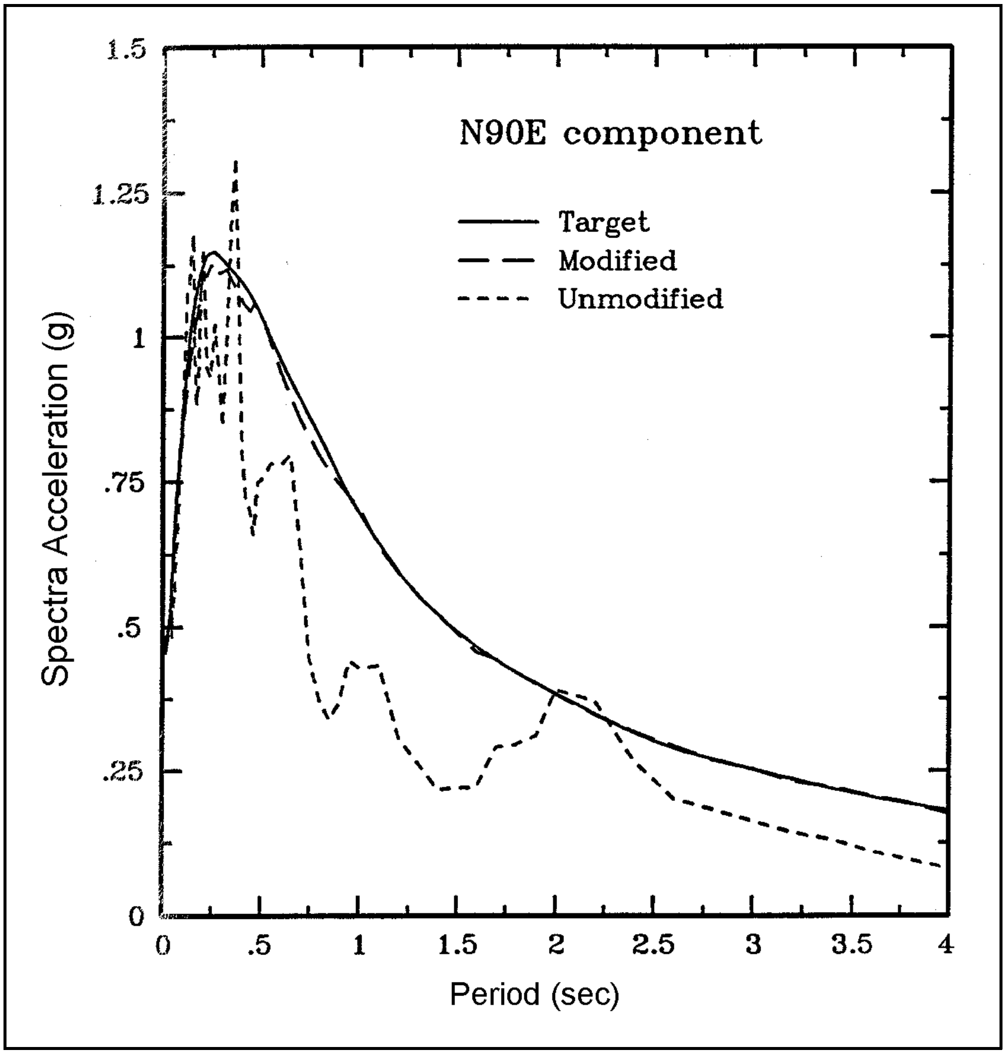


Figure 3-18. Example of response spectrum of time-history matched to a design response spectrum and spectrum of original time-history

The disadvantage is that the motions are not "real" motions, which would not exhibit smooth design spectra. Also "real" motions may contain less energy than synthetic spectrum-matched motions of similar amplitude. Although a good fit to a design spectrum can be attained with a single accelerogram, it may be desirable in some cases to fit the spectrum using more than one accelerogram. For nonlin ear analysis applications, it is particularly desirable to have multiple accelerograms because different accelerograms may have different pulse sequencing characteristics of importance to nonlinear response yet have essentially identically response spectra. Numerical ground motion modeling methods can also be used to produce synthetic accelerograms. Such motions have the character of recorded motions since the modeling procedures are intended to simulate the earthquake rupture and wave propagation process.

3-7. Developing Site-Specific Spectra for Soil Sites

As is the case for developing site-specific rock spectra, either the approach of anchoring spectral shapes to a peak ground acceleration (Approach 1) or the approach of directly estimating the spectra (Approach 2) can be used for soil sites. The implementation of these approaches is outlined below.

a. Approach 1 - Anchoring the response spectral shape to the peak ground acceleration.

(1) Estimating peak ground acceleration. Table 3-1 summarizes recently developed attenuation relationships for estimating peak ground acceleration. For WUS shallow crustal faulting earthquakes, several recent attenuation relationships are available to estimate top-of-soil peak ground accelerations for firm soil conditions, and Idriss (1991a) has developed a peak acceleration attenuation relationship for soft soil sites. Combining the BLWN/RVT method for rock motion estimation with a site response analysis for a deep soil column, Boore and Joyner (1991) developed a peak ground acceleration attenuation relationship for deep soil sites in the EUS. For subduction zone earthquakes, attenuation relationships have been developed for firm soil conditions and in some cases for soft soil conditions (Table 3-1). As is the case for rock attenuation relationships, the soil attenuation relationships are better constrained by data for WUS shallow crustal earthquakes than for EUS or subduction zone earthquakes.

(2) Estimating response spectral shape and the response spectrum. Spectral shapes that have been developed for soil sites for WUS shallow crustal earthquakes and for subduction zone earthquakes on the basis of statistical analyses of ground motion data are summarized in Table 3-2. Spectral shapes have not been developed for soil sites for EUS earthquakes. Using the Newmark and Hall (1978, 1982) approach (Appendix B), the effect of soil is accounted for by estimating values of peak ground velocity and peak ground displacement directly, or by using Newmark and Hall's relationships between peak ground acceleration, peak ground velocity, and peak ground displacement for firm soil to estimate peak ground velocity and peak ground displacement, given peak ground acceleration. Peak ground acceleration, velocity, and displacement then are multiplied by Newmark and Hall's amplification factors to obtain the absolute response spectrum.

(3) Uncertainty in response spectra predictions. The comments made in paragraph 3-6*a*(3) regarding estimating 84th percentile response spectra for rock also apply to response spectra for soil.

(4) Estimating vertical ground motion response spectra. Recent studies (e.g., Silva 1997) indicate that vertical-to-horizontal ratios of response spectra of ground motions are higher on soil than on rock for short periods of vibration. Figure 3-16 provides a guideline for ratios of vertical-to-horizontal spectral ratios on firm soil sites that is generally conservative. However, if a facility is sensitive to short-period (less than 0.3 sec) vertical motions, and is located within 25 km of the earthquake source or the magnitude exceeds 6.5, further evaluation of vertical response spectra on soil is recommended because vertical response spectra can significantly exceed horizontal response spectra for these conditions.

b. Using Approach 2 - Estimating the soil response spectrum directly.

(1) General. The three approaches discussed in paragraph 3-5*b* can be used for soil sites. These approaches are using response spectral attenuation relationships, performing statistical analysis of ground motion response spectra, and theoretical (numerical) modeling.

(2) Using response spectral attenuation relationships. Recently developed response spectral attenuation relationships for soil are summarized in Table 3-3. The spectral attenuation relationships in

Table 3-3 generally include the standard deviations of the estimates, from which 84th percentile values may be obtained.

(3) Using statistical analyses of response spectra. Abundant ground motion data for firm soil sites in the WUS are available to permit statistical analyses of data sets for many design earthquakes. Although such data are not available for EUS earthquakes, the WUS data can be used, recognizing that the analyses may underestimate short-period (less than 0.1 to 0.3 sec) response spectra. (Refer to discussion for rock sites in paragraph 3-6*b*(3).) The degree of underestimation should be less at soil sites than at rock sites because soils will tend to damp out the short-period motions.

(4) Using theoretical (numerical) modeling techniques. The techniques discussed in paragraph 3-5*b*(3) can be used to simulate the earthquake rupture and propagation of seismic waves from the earthquake source to the site. In addition, site response analyses may be carried out to estimate top-of-soil response spectra, given a response spectrum in rock. With this approach, rock motions, including response spectra, are first defined for the site (using procedures and relationships described in paragraph 3-6). The soil profile between the ground surface and the underlying rock is modeled. The rock motions are assigned to a hypothetical rock outcrop at the site rather than to the rock at depth beneath the soil column. This is because rock motion recordings are obtained at the ground surface rather than at depth, and unless the rock is rigid, the rock motion beneath the soil column will differ somewhat from the rock outcrop motion. Then, using nonlinear or equivalent linear soil response analytical methods, rock motions are propagated through the soils column and top-of-soil motions are estimated. The site response analysis process is schematically illustrated in Figure 3-19. This figure illustrates the commonly used one-dimensional site response analysis method applicable where the soil stratigraphy is relatively uniform and flat-lying and the ground surface topography is relatively level. Two-dimensional site response analysis methods are available for situations where these conditions are not sufficiently met. Just as in other types of theoretical modeling and numerical analyses, results of site response analyses are sensitive to the details of the analytical procedures, modeling of the process, and inputs to the analysis. Broad guidelines for conducting these analyses are listed below:

(a) More than one input rock acceleration time-history should be used, and the selected motions should be reasonably representative of the rock motions in terms of spectra and duration.

(b) Parametric analyses for variations in the dynamic soil properties should be made to examine the sensitivity of the response to uncertainties in the soil properties. This is particularly important when soil properties are based on generalized correlations rather than on a program of field shear wave velocity measurements.

(c) It is useful to compute ratios of response spectra of top-of-soil motion to input rock motion for each analysis that is carried out. The ratios are much less sensitive to the actual input motion than is the absolute top-of-soil motion. The spectral ratios can then be examined and smoothed and multiplied by the rock smooth spectrum to obtain a top-of-soil spectrum, which can be further smoothed.

An illustration of a site response analysis is presented in Appendix E. Articles by Seed and Sun (1989), Chang et al. (1990), and Ahmad, Gazetas, and Desai (1991) provide useful background information on site response analysis methodologies. Site response analyses are needed relatively more for soft soil sites than for firm soil sites because the site response effects are greater for soft soils and because ground motion data and empirically based ground motion relationships are relatively scarce for soft soil sites.

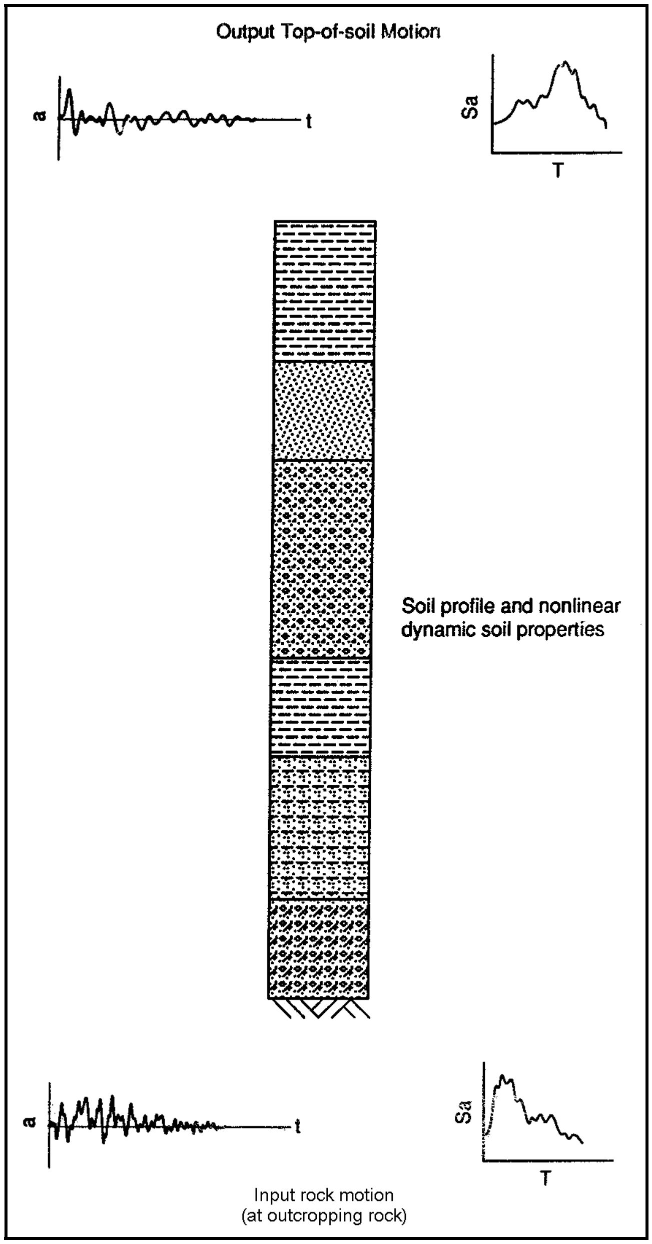


Figure 3-19. Schematic of one-dimensional site response analysis

(5) Estimating vertical ground motion response spectra. The available recently developed attenuation relationships for response spectral accelerations of vertical firm-soil site motions are summarized in Table 3-3. These relationships can be used directly to estimate vertical response spectra on firm soils sites. The vertical to horizontal response spectral ratios discussed in paragraph 3-7*a*(4) and shown in Figure 3-16 can be used as a guide in estimating vertical response spectra, given an estimate of horizontal response spectra.

c. Developing acceleration time-histories of soil motions consistent with design response spectrum. The two alternatives for developing acceleration time-histories for rock motions that were discussed in paragraph 3-6*c*, namely, selecting recorded motions and synthetically developing or modifying motions, can also be used to develop time-histories for top-of-soil motions. In the case where site response analyses are carried out to define top-of-soil motions, there is a third alternative, which is to obtain the time-histories directly from the site response analyses.

Section III
Probabilistic Approach for Developing Site-Specific Response Spectra

3-8. Overview of Probabilistic Seismic Hazard Analysis (PSHA) Methodology

a. General. PSHA takes the elements of a deterministic assessment of earthquake ground shaking hazard—identification of seismic sources; specification of limiting earthquake sizes; assessment of ground motions as a function of earthquake magnitude; source-to-site distance; and site conditions—and adds an assessment of the likelihood that ground shaking will occur during a specified time period. Figure 3-20 shows a typical result of PSHA, termed a hazard curve, that relates the level of ground shaking to the annual frequency of exceedance of that level. The ground motion parameter for the example in Figure 3-20 is peak ground acceleration. PSHA may be conducted for other ground motion parameters, such as peak ground velocity or response spectral values for specific periods of vibration and damping ratios. If a PSHA is carried out for response spectral values at a number of periods of vibration, then response spectra having selected probabilities of exceedance (i.e., “equal hazard” response spectra) may be constructed, as will be discussed later.

b. Elements of a PSHA. Evaluation of the frequency or probability of exceedance of ground motions at a site is a function of earthquake source definition (distance of the sources from the site, source geometries, and frequencies of occurrence (recurrence) of earthquakes of different magnitudes on each source), and ground motion attenuation (amplitudes of ground motion as a function of earthquake magnitude and distance). These basic inputs to a PSHA are then combined in a probabilistic model to obtain hazard curves and (if desired) equal-hazard response spectra as discussed above. The basic elements of a PSHA are illustrated in Figure 3-21 for peak ground acceleration and in Figure 3-22 for equal hazard response spectra.

c. Formulation of PSHA methodology.

(1) Formulation. The methodology used to conduct PSHA was initially developed by Cornell (1968). Current practice is described in several publications, such as National Research Council (1988) and Earthquake Engineering Research Institute Committee on Seismic Risk (1989). Using a Poisson probability model (paragraph 3-9*d*), the probability of exceedance $p_E(z)$ of a ground motion level z in an exposure time or design time period t at a site is related to the annual frequency of ground motion exceedance at the site, ν_z, by:

$$p_E(z) = 1 - e^{-(\nu_z \cdot t)} \tag{3-1}$$

A PSHA is carried out to obtain ν_z, and $p_E(z)$ can then be obtained using Equation 3-1. The return period (RP) for ground motion exceedance at a site is equal to the reciprocal of ν_z. The results of a PSHA are, in practice, expressed in terms of one or more of the parameters, $p_E(z)$, ν_z, and RP. Using Equation 3-1, the interrelationship between these fundamental parameters is illustrated in graphical form in Figure 3-23 and in tabular form in Table 3-4. Note that when ($\nu_z \bullet t$) is small (approximately $\leq$ 0.1), $p_E(z)$ is approximately equal to $\nu_z \bullet t$. For larger values of $\nu_z \bullet t$, $p_E(z)$ is less than ($\nu_z \bullet t$). The basic formulation for ν_z is:

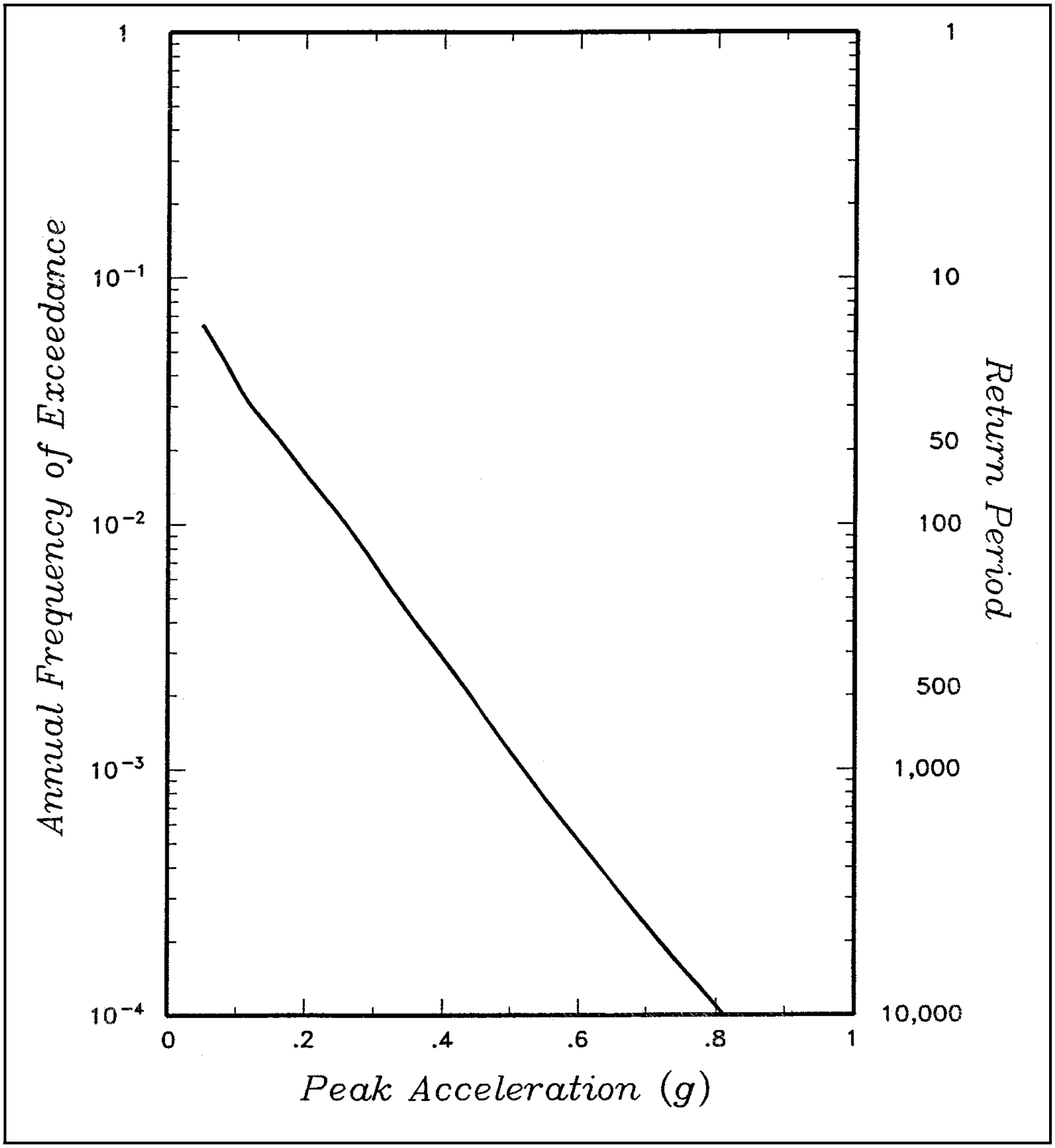


Figure 3-20. Example seismic hazard curve showing relationship between peak ground acceleration and probability (annual frequency) of exceedance

$$v_z = \sum_N \left[\sum_M \lambda\,(m_i) \bullet \sum_R P\,(R = r_j\,|m_i) \bullet P\,(Z > z\,|\,m_i\ r_j) \right]_n \tag{3-2}$$

where

$\sum_N$ = summation over all (N) seismic sources

$\lambda(m_i)$ = the annual frequency of occurrence of earthquakes of magnitude m_i (above a certain minimum size of engineering significance) on seismic source n

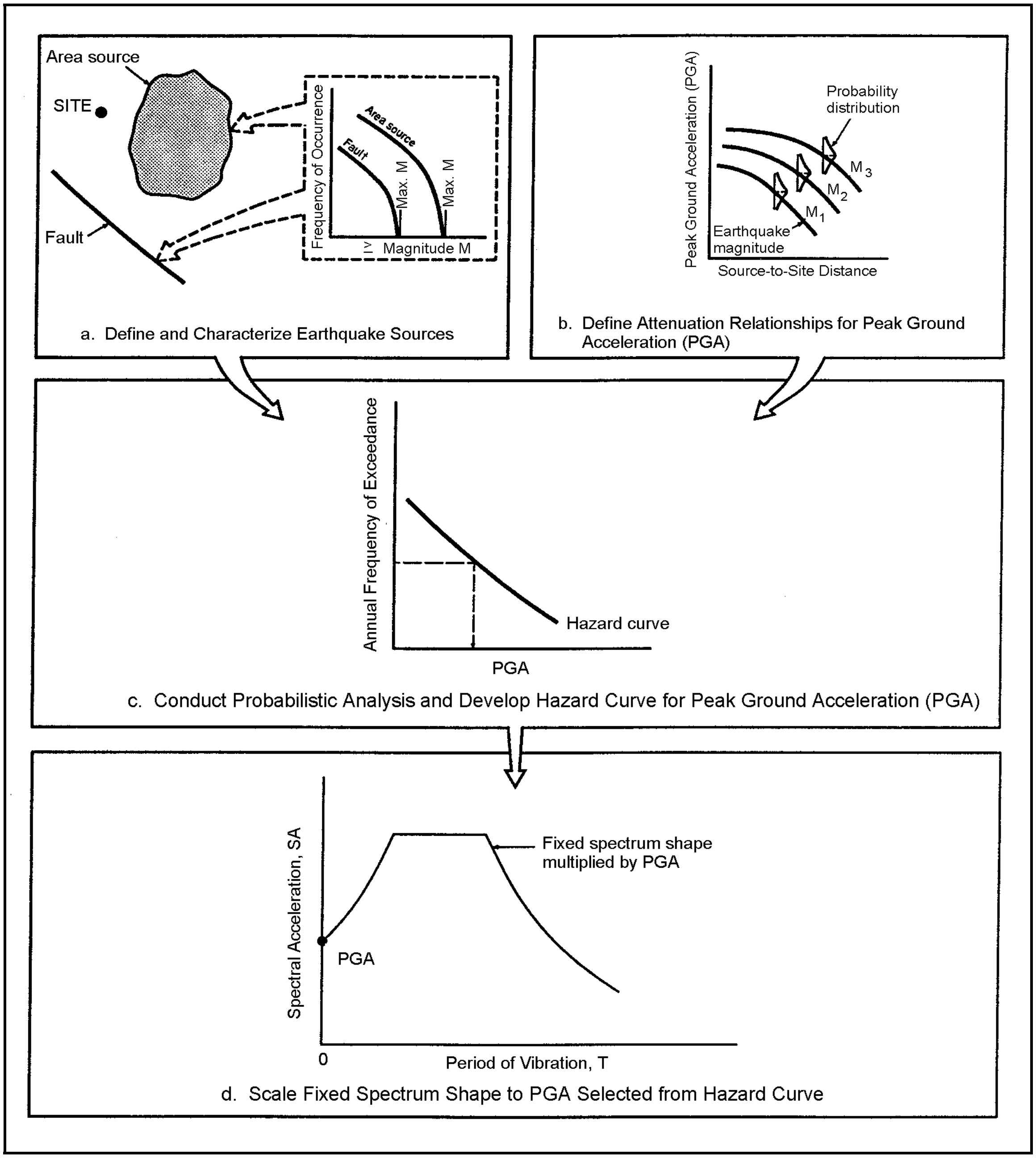


Figure 3-21. Development of response spectrum based on a fixed spectrum shape and a probabilistic seismic hazard analysis for peak ground acceleration

$P\,(R=r_j \mid m_i)$ = the probability of an earthquake of magnitude m_i on source n occurring at a certain distance r_j from the site

$P\,(Z>z \mid m_i, r_j)$ = the probability that ground motion level z will be exceeded, given an earthquake of magnitude m_i on source n at distance r_j from the site

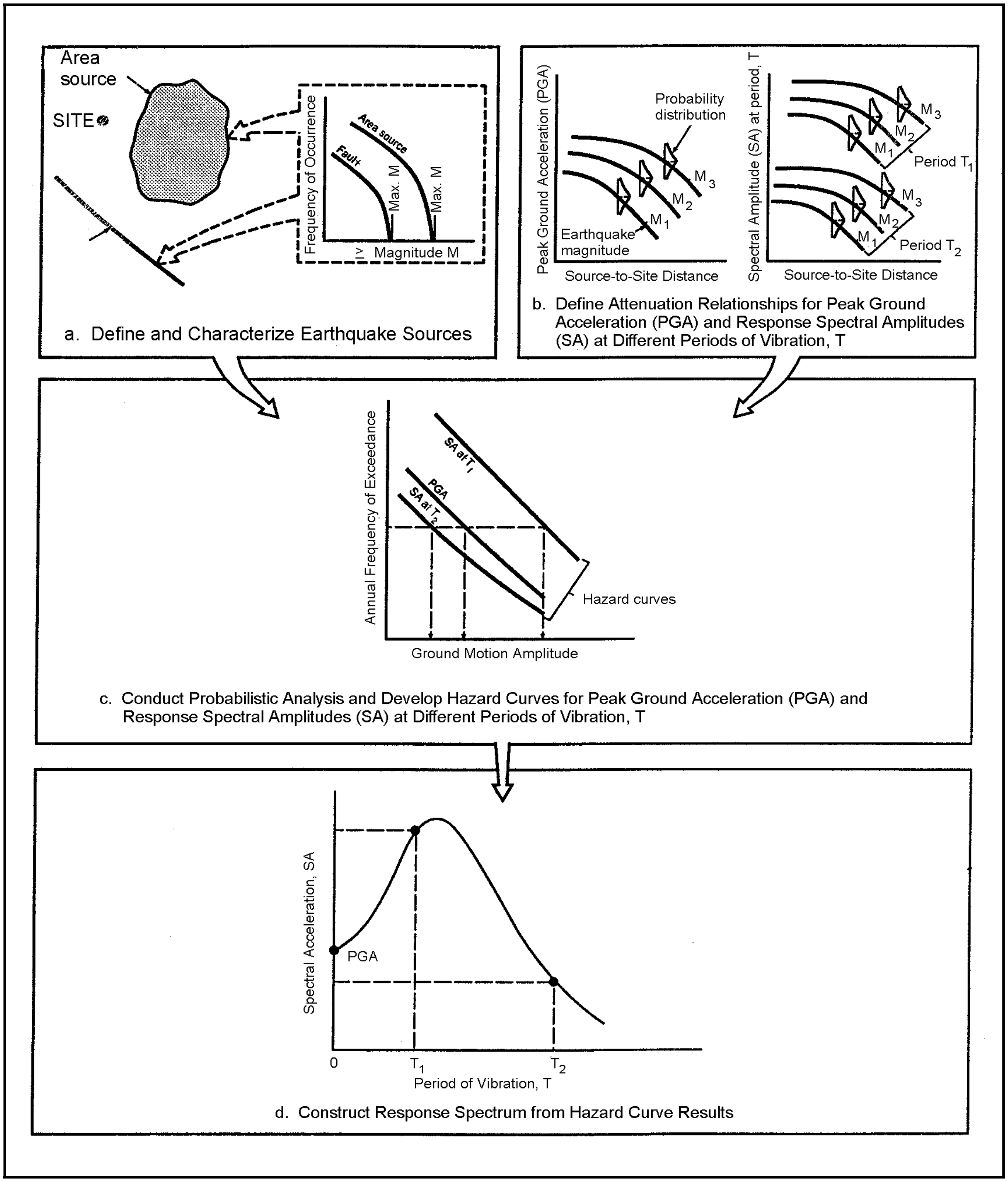


Figure 3-22. Development of equal-hazard response spectrum from probabilistic seismic hazard analysis for response spectral values

Thus, for a given source, the annual frequency or rate of exceeding a certain ground motion level at the site is obtained by summing over all magnitudes and source-to-site distances for that source. Then, the total rate of ground motion exceedance at the site ν_z is obtained by adding the rates for all the sources. The components of Equation 3-2 are discussed in (2), (3), and (4) below. A simplified example of a PSHA illustrating the calculation process using Equation 3-2 is presented in Appendix G (Example G-1).

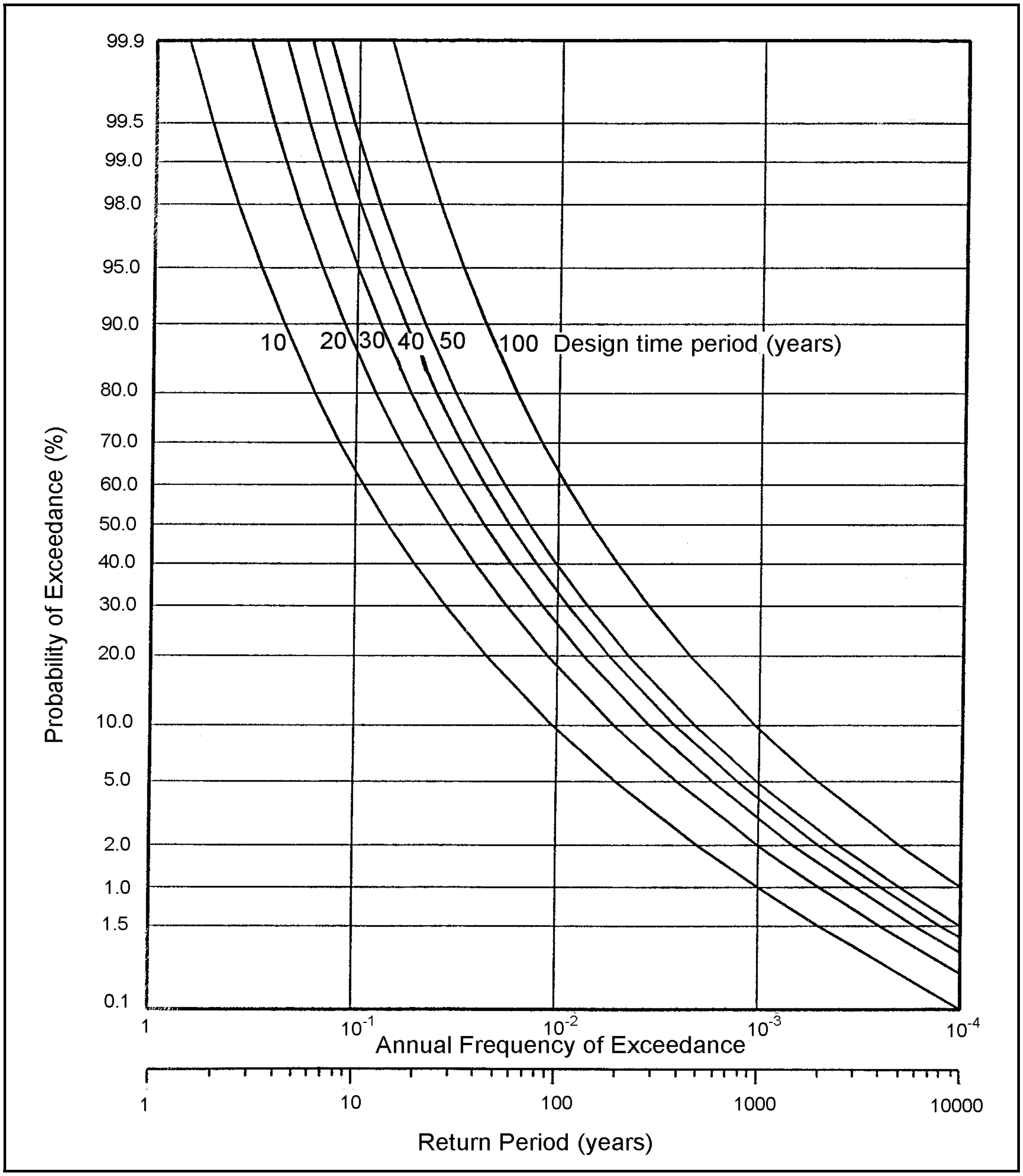


Figure 3-23. Relationship between annual frequency of exceedance/return period and probability of exceedance for different design time periods

(2) Rate of occurrence of earthquakes. The rate of occurrence of earthquakes $\lambda(m_i)$ is obtained based on earthquake recurrence assessments. Typical earthquake recurrence curves for earthquake sources are illustrated in the upper part of Figure 3-24. As shown, recurrence curves express the rate of occurrence of earthquakes equal to or greater than a certain magnitude. $\lambda(m_i)$ is obtained by discretizing the recurrence curves into narrow magnitude intervals as illustrated in the lower part of Figure 3-24. The two different types of magnitude distributions shown in Figure 3-24, exponential and characteristic, are discussed in paragraph 3-9(d).

Table 3-4
Relationship Between Return Period and Probability of Exceedance for Different Time Periods

	Return Period, Years, for Different Design Time Periods *t*					
Probability of Exceedance, %	***t* = 10 years**	***t* = 20 years**	***t* = 30 years**	***t* = 40 years**	***t* = 50 years**	***t* = 100 years**
1	995	1,990	2,985	3,980	4,975	9,950
2	495	990	1,485	1,980	2,475	4,950
5	195	390	585	780	975	1,950
10	95	190	285	380	475	950
20	45	90	135	180	225	450
30	28	56	84	112	140	280
40	20	39	59	78	98	195
50	14	29	43	58	72	145
60	11	22	33	44	55	110
70	8.3	17	25	33	42	83
80	6.2	12	19	25	31	62
90	4.3	8.7	13	17	22	43
95	3.3	6.7	10	13	17	33
99	2.2	4.3	6.5	8.7	11	22
99.5	1.9	3.8	5.7	7.5	9.4	19

(3) Distance probability distribution. The distance probability distribution, $P(R=r_j \mid m_i)$, depends on the geometry of earthquake sources and their distance from the site; an assumption is usually made that earthquakes occur with equal likelihood on different parts of a source. The function $P(R=r_j \mid m_i)$ also should incorporate the magnitude-dependence of earthquake rupture size; larger magnitude earthquakes have larger rupture areas, and thus have higher probability of releasing energy closer to a site than smaller magnitude earthquakes on the same source. An example of probability distributions for the closest distance to an earthquake source is shown in Figure 3-25. In this particular example, the source (fault) is characterized as a line source, and the probability distributions are based on the formulations presented by Der Kiureghian and Ang (1977). Figure 3-25a illustrates the probability distributions for a fault rupture length of 5 km; Figure 3-25b illustrates the probability distributions for a fault rupture length of 25 km. The longer rupture length corresponds to a larger magnitude. The figure shows the distributions for both the probability of the closest distance to the fault rupture R being less than a certain value $P(R<r_j \mid m_i)$, and the probability of earthquakes occurring at a certain distance ($P(R=r_j \mid m_i)$, which is obtained by discretizing the curves for $P(R<r_j \mid m_i)$. The higher probability for earthquakes to occur at closer distances for longer rupture lengths (larger magnitudes) can be noted by comparing Figure 3-25b with 3-25a. It can also be observed in Figure 3-25 that there is zero probability of earthquake occurrence either closer than the closest distance to the earthquake source (10 km in the example) or farther than the closest distance to the rupture placed at the farthest end of the fault (farther than a distance of approximately 61 km $[10^2 + (65 - 5)^2]^{1/2}$ for a 5-km rupture and a distance of approximately 41 km $[10^2 + (65 - 25)^2]^{1/2}$ in the case of the 25-km rupture length). The probability abruptly changes at a closest distance equal to the distance defined by placing the fault rupture at the nearest end of the fault (distance of approximately 32 km $[10^2 + (35 - 5)^2]^{1/2}$ for a 5-km rupture and a distance of approximately 14 km $[10^2 + (35 - 25)^2]^{1/2}$ for a 25-km rupture). Note that the distance to the earthquake rupture must be

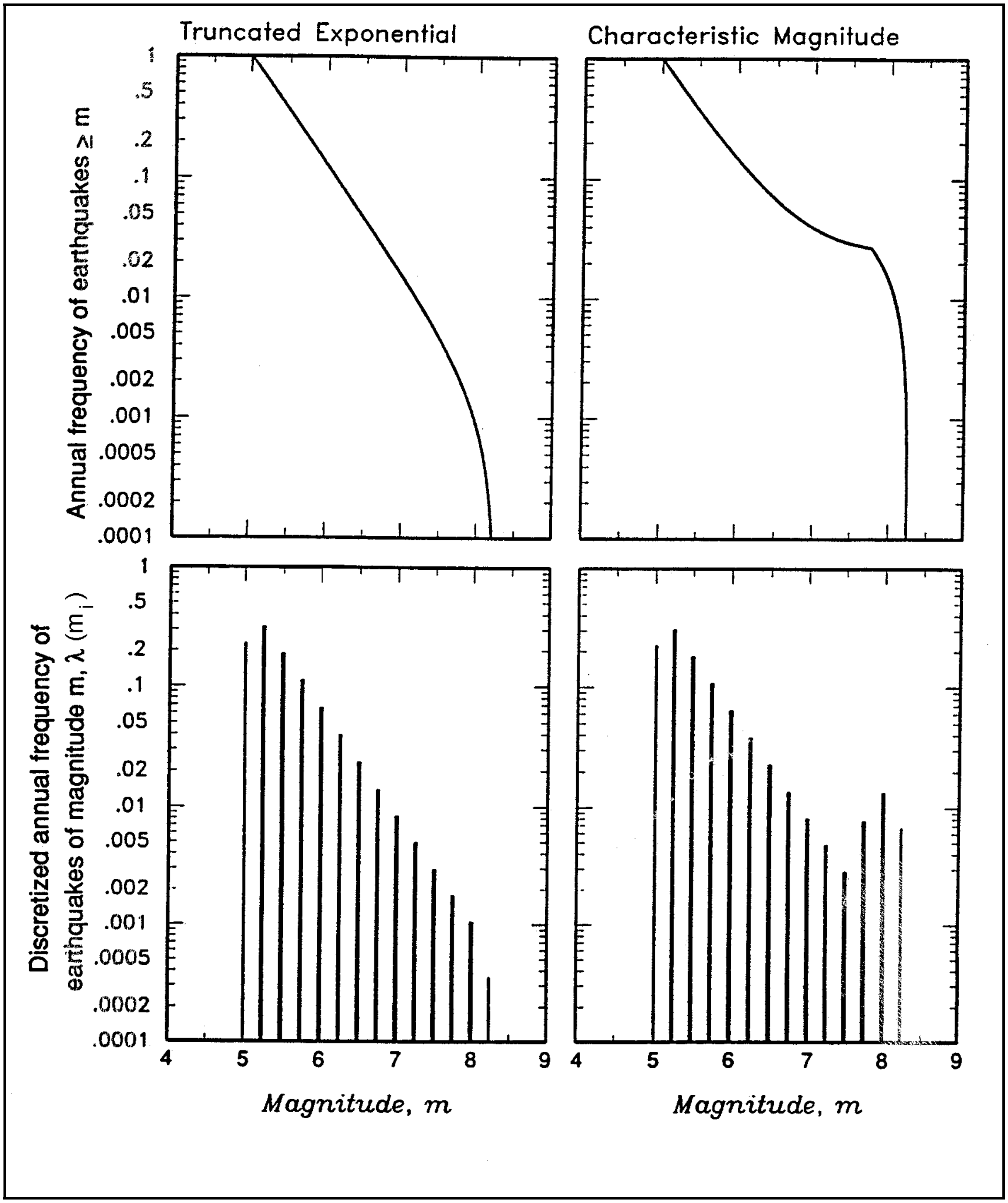


Figure 3-24. Typical earthquake recurrence curves and discretized occurrence rates

expressed in terms of the same definition of distance as used in the ground motion attenuation relationships. Typically, some form of closest distance to rupture definition is used for attenuation relationships (variations in this definition include closest distance to rupture, closest distance to rupture of the seismogenic zone, closest horizontal distance to surface projection of rupture, etc.).

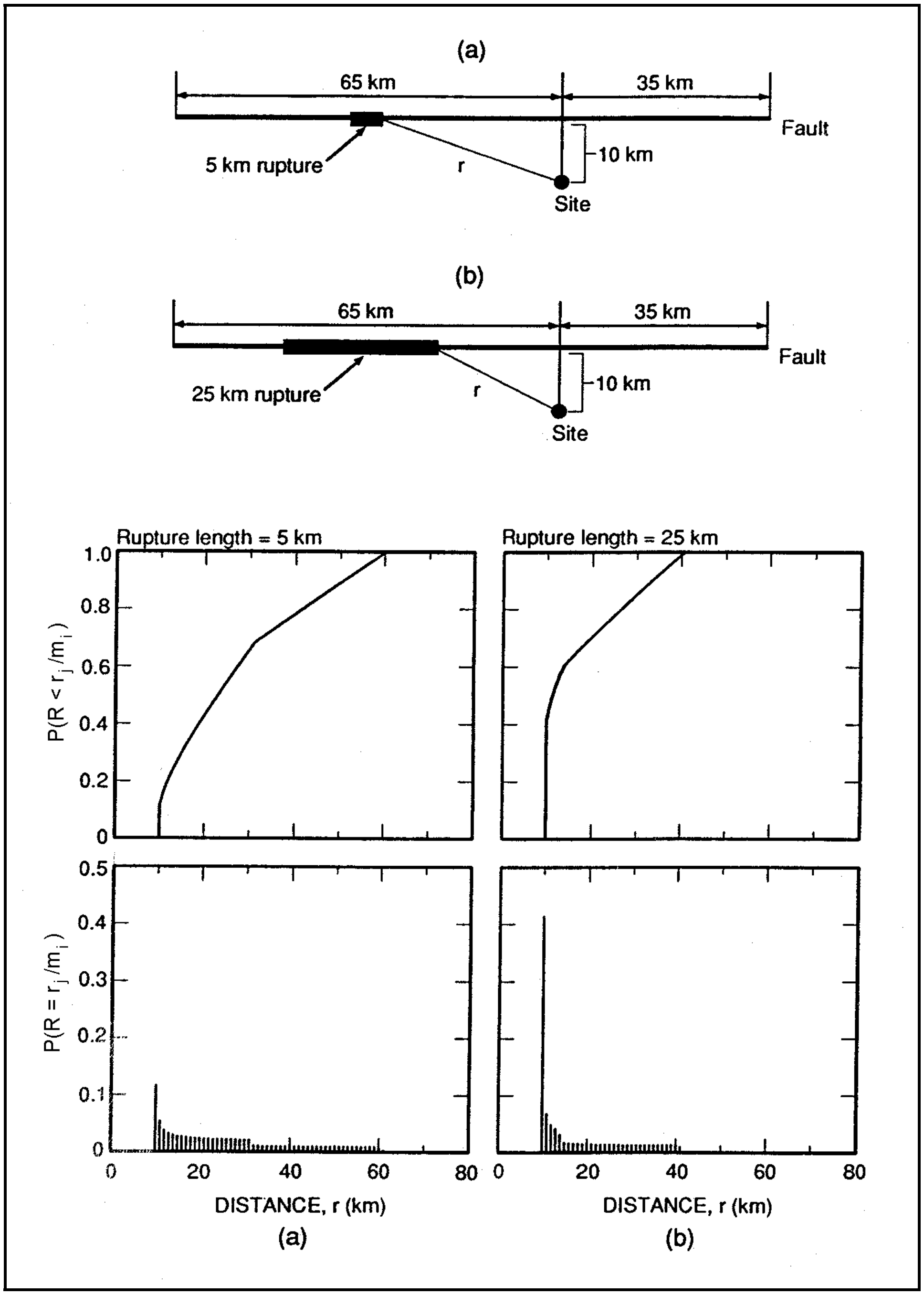
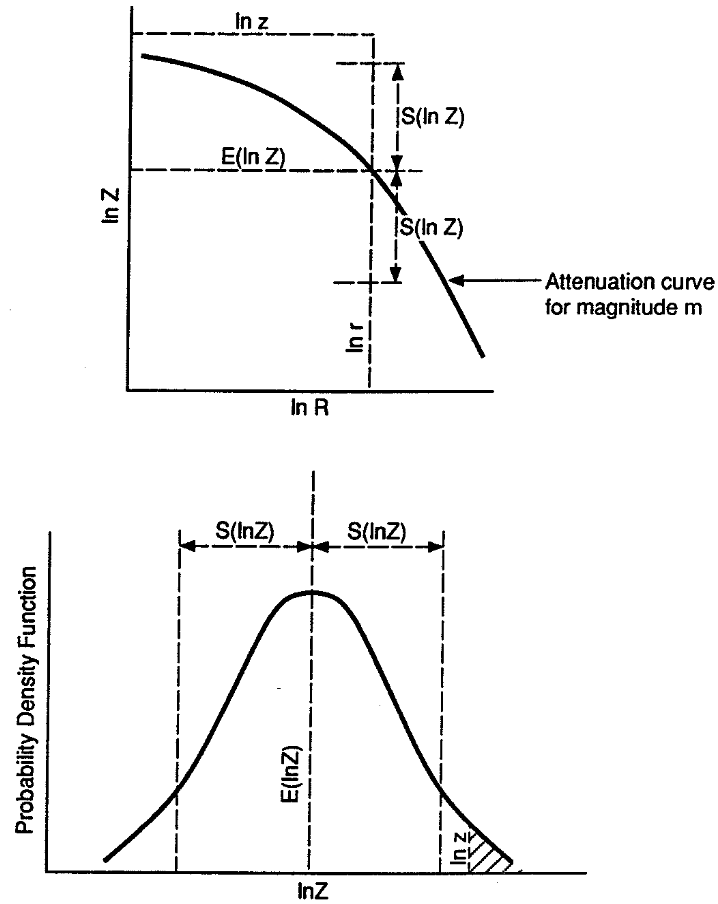


Figure 3-25. Illustration of distance probability distribution

(4) Conditional probability of ground motion exceedance. The conditional probability of exceeding a ground motion level for a certain earthquake magnitude and distance $P(Z>z|m,r)$ is determined from the ground motion attenuation relationships selected for the site. (Available relationships are discussed in Section II.) These relationships incorporate the uncertainty in ground motion estimation given m and r (see Figures 3-21 and 3-22). The function $P(Z>z|m,r)$ is usually evaluated assuming that ground motion values are log normally distributed about the median value; the calculation of this function is illustrated in Figure 3-26.



$$P(Z>z \mid m, r) = 1 - F_u \left(\frac{\ln z - E(\ln Z \mid m, r)}{S(\ln Z \mid m, r)} \right)$$

EXPLANATION

E(lnZ) is the expected value of lnZ

S(lnZ) is the standard deviation of the estimate of lnZ

F_u is cumulative distribution function of a unit normal variable

Figure 3-26. Ground motion estimation conditional probability function

3-9. Characterizing Seismic Sources for PSHA

Although the following discussion is oriented toward probabilistic approaches for characterizing seismic sources, much of the discussion is applicable to deterministic approaches as well.

a. Source identification.

(1) Seismic source. A seismic source represents a region of the earth's crust where the characteristics of earthquake activity are recognized to be different from those of the adjacent crust. Seismic sources are identified on the basis of geological, seismological, and geophysical data. An understanding of the regional tectonics, local Quaternary geologic history, and seismicity of an area leads to the identification of seismic sources. The development of tectonic models for crustal deformation and the assessment of the tectonic role of individual geologic structures are useful for both identifying potential sources and assessing their characteristics. Geologic studies can be used to assess the location, timing, and style of crustal deformation. The association of geologic structures with historic or instrumental seismicity may clarify their role within the present tectonic stress regime. Characteristics of seismicity, including epicenter locations, focal depths, and source mechanisms, also aid in identifying potential sources.

(2) Faults. Because earthquakes occur as a result of differential slip on faults, modeling of seismic sources as individual faults is the most physically realistic model for seismic hazard analysis. Under favorable conditions, individual faults can be identified and treated as distinct seismic sources. Active faults are usually identified on the basis of geomorphic expression and stratigraphic displacements but can also be identified by lineations of seismicity, by geophysical measurements, or by inference from detailed investigations of related geologic structures, such as active folding or crustal plate subduction. A fault model for individual sources allows the use of geologic data on fault behavior, as well as seismicity data, to characterize earthquake activity.

(3) Seismic source zones. In areas with low rates of crustal deformation away from plate margins, such as the EUS, seismic sources are often defined as seismic source zones. Seismic source zones are used to model the occurrence of seismicity in areas where specific faults cannot be identified and where the observed seismicity exhibits a diffuse pattern not clearly associated with individual faults. These conditions are typical of areas with lower rates of crustal deformation, such as regions away from plate margins (e.g., EUS). Seismic source zones can be defined based either on historical seismicity patterns or geology and tectonics. When defined based on historical seismicity patterns, a large region can be subdivided into small regular areas that are treated as individual source zones (Electric Power Research Institute (EPRI) 1987; U.S. Geological Survey (USGS) 1996). With this approach, it is assumed that the spatial variation in the occurrence rate of future earthquakes is similar to the historical pattern of seismicity. Due to the relatively short historical period and low rates of seismicity, the seismicity patterns are usually determined by small earthquakes. It is not clear whether this pattern reflects the likelihood of future earthquakes of engineering significance (generally taken to be earthquakes of magnitude approximately equal to or greater than 5). The alternative approach is to define areas thought to have homogeneous earthquake potential characteristics (in terms of rate of earthquake occurrence and maximum earthquake size) on the basis of the geology and tectonics of the region. For example, recent evidence from studies of global earthquakes in stable continental regions such as the EUS has shown that most larger earthquakes occur through reactivation of faults in geologically ancient rift zones (Johnston et al. 1994). Where available, paleoseismological data (e.g., spatial and temporal distribution of liquefaction features) should be used to identify source regions for large-magnitude earthquakes. Because of uncertainty regarding the most appropriate model for earthquake occurrence, both seismicity-based and geologically based seismic source zones should be included in a probabilistic analysis.

b. Source geometry.

(1) General. Description of the geometry of a seismic source is necessary to evaluate the distances from the site at which future earthquakes could occur. In addition, source geometry can place physical constraints on the maximum size earthquake that can occur on a source.

(2) Faults. Seismic sources defined as faults are modeled in a PSHA as segmented linear or planar features. Earthquake ruptures on fault sources are modeled as rupture lengths or rupture areas, with the size of rupture defined on the basis of empirical relationships between earthquake magnitude and rupture size (Wyss 1979; Wells and Coppersmith 1993).

(3) Seismic source zones. For seismic sources defined as geologic structures suspected to contain faults, the distribution of earthquakes can be modeled as rupture surfaces occurring on multiple fault planes distributed throughout the source volume if the general trend of such planes is known or can be inferred. Alternatively, earthquake locations can be modeled as random point sources within the source volume if the orientation of potential fault planes is unknown. The spatial distribution of seismicity within large areal sources can be modeled similarly.

c. Maximum earthquake magnitude.

(1) Faults. The limiting size earthquake that can occur on each seismic source is an important parameter, especially in evaluating seismic hazard at low probability levels. The maximum magnitude can most easily be estimated when the seismic source is defined on the basis of an identifiable fault. For faults, the maximum earthquake magnitude is related to fault geometry and fault behavior through an assessment of the maximum dimensions of a single rupture. Evaluation of fault segmentation can play a key role in identifying portions of a fault zone likely to represent the largest size of a single rupture (Schwartz and Coppersmith 1986; Schwartz 1988). The maximum magnitude is related to the maximum rupture size through empirical relationships (Slemmons 1982; Bonilla, Mark, and Lienkaemper 1984; Wells and Coppersmith 1994). Because these relationships are subject to uncertainty, the use of a number of magnitude estimation techniques can result in more reliable estimates of maximum magnitude than the application of a single relationship.

(2) Seismic source zones. The assessment of maximum magnitude is more difficult when seismic sources are defined on the basis of large-scale tectonic features or crustal blocks, as is typically done in the EUS. In such cases the maximum magnitude is often estimated to be the maximum historical earthquake magnitude plus an increment, or is estimated to be a magnitude having a specified return period. The chief weakness of these approaches is the generally short period of historical observations compared with the likely return period of a maximum event for an individual source. Another approach that attempts to extend the generally short observational period for individual sources is based in augmenting the assessment using data from analogous structures worldwide. This approach identifies analogous features for which the maximum magnitude is better defined or identifies the largest event that has occurred on such features. In the application of analogies, the seismicity of similar structures on a worldwide basis can be examined to supplement the limited local historical record. Recent efforts have been made to use a global earthquake database to identify the factors that control or limit the maximum size of earthquakes within stable continental regions like the EUS to develop a formal method for estimating maximum magnitude in such regions (Johnston et al. 1994)

d. Rate of earthquake occurrence and distribution in earthquake size.

(1) Estimating recurrence rates. Earthquake recurrence rates are estimated from historical seismicity, from geological data on rates of fault movement, and from paleoseismic data on the timing of large prehistoric events. For areal sources, historical seismicity is usually used to estimate earthquake recurrence rates. When recurrence for small, regular source zones (cells) is analyzed (*a*(3) above), procedures can be employed to smooth seismicity rates among adjoining cells (EPRI 1987; USGS 1996). In an analysis of the earthquake catalog of historical seismicity, it is important to translate the data into a common magnitude scale consistent with the magnitude scale used in the ground motion attenuation

relationships, and to account for completeness in earthquake reporting as a function of time and location. Straightforward statistical techniques can then be used to estimate earthquake recurrence parameters (Weichert 1980).

(2) Use of geologic data for faults. For sources defined as individual faults, the available historical seismicity is usually insufficient to characterize the earthquake recurrence. Geologic data on fault slip rates can be used to estimate the rate of seismic moment release, leading to the rate of earthquake recurrence. In addition, paleoseismic studies of the occurrence of large prehistoric events can be used to estimate recurrence of larger magnitude earthquakes on a fault. Predictions of recurrence rates for larger events from fault-specific geologic data have been shown to match well with observed historical rates on a regional basis (Youngs and Coppersmith 1985b; Youngs, Swan, and Power 1988; Youngs et al. 1987; Youngs et al. 1993b). The rate of earthquake occurrence may not be uniform along the strike or dip of an earthquake fault. Evaluation of fault segmentation can be used to characterize variations in recurrence along the length of a fault. The depth distribution of historical seismicity can be used to specify down-dip variations in recurrence.

(3) Recurrence models. The relative frequency of various size earthquakes has usually been specified by the truncated exponential recurrence model (Cornell and Van Marke 1969) based on Gutenberg and Richter's (1954) recurrence law. This model was developed on the basis of observations of global seismicity. It has been found to work well on a regional basis and for modeling seismicity for nonfault specific sources such as distributed seismicity zones and generalized tectonic structures. Recent advances in understanding of the earthquake generation process have indicated that earthquake recurrence on individual faults may not conform to the exponential model. Instead, individual faults or fault segments may tend to rupture in what have been termed “characteristic” magnitude events at or near the maximum magnitude (Schwartz and Coppersmith 1984). This has led to the development of fault-specific recurrence models such as the maximum moment model (Wesnousky et al. 1983) and the characteristic magnitude recurrence model (Youngs and Coppersmith 1985a, 1985b). Figure 3-27 illustrates a characteristic magnitude recurrence model. Figure 3-28 compares exponential and characteristic earthquake recurrence relationships. The figure illustrates the differences between the two recurrence models depending on how the earthquake recurrence rate is specified. Figure 3-28a shows recurrence curves if the total rate of seismic moment release is specified to be the same for each model; Figure 3-28b shows recurrence curves if the rate of large-magnitude earthquakes is specified to be the same for each model. Detailed studies of earthquake recurrence in the Wasatch fault region, Utah, and in the San Francisco Bay region have shown excellent matches between regional seismicity rates and recurrence modeling when combining the characteristic recurrence model for individual faults with the exponential model for distributed source areas (Youngs et al. 1987; Youngs, Swan, and Power 1988; Youngs et al. 1993b).

(4) Poisson versus real-time recurrence. Nearly all PSHA's assume that earthquake occurrence in time is a random and memoryless (Poisson) process. In the Poisson process, it is assumed that the probability of an event in a specified period is completely determined by the average frequency of occurrence of events, and the probability of occurrence of the next event is independent of when the last event occurred. While the observed seismicity data on a regional basis have been shown to be consistent with the Poisson model, the model does not conform to the physical process believed to result in earthquakes, one of a gradual, relatively uniform rate of strain accumulation followed by sudden release. Detailed paleoseismic studies of several faults as well as historical seismicity from very active subduction zones have indicated that the occurrence of the larger events on a source tends to be more cyclic in nature. These observations have led to the use of nonstationary or “real-time” recurrence models that predict the probability of events in the next period, rather than any period. Typically, a

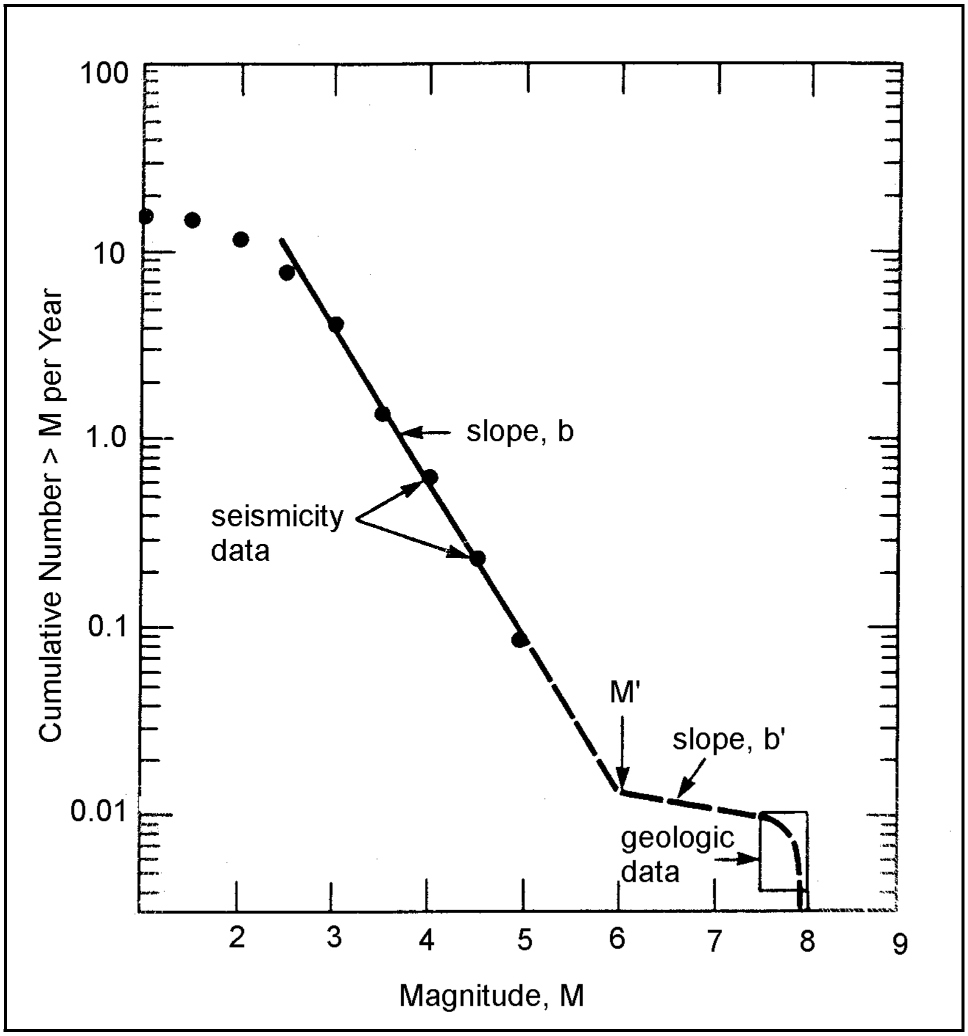


Figure 3-27. Diagrammatic characteristic earthquake recurrence relationship for an individual fault or fault segment. Above magnitude *M*' a low *b* value (*b*') is required to reconcile the small-magnitude recurrence with geologic recurrence, which is represented by the box (Schwartz and Coppersmith 1984; National Research Council 1988)

simple "renewal model" is used to evaluate the likelihood of events within specified future periods. A recent example of the use of a renewal model was a study of the probabilities of large earthquakes on the San Andreas fault system in northern California for use in regional planning (Working Group on California Earthquake Probabilities 1990). Consideration may be given to such time-dependent models in the few cases where there is sufficient information to develop the required parameters.

3-10. Ground Motion Attenuation Characterization for PSHA

Specification of ground motions for PSHA is subject to all of the requirements discussed in Section II for deterministic ground motion assessments. The analysis requires ground motion attenuation relationships for the full range of magnitudes and distances considered. Recently developed attenuation relationships for peak ground acceleration and response spectral values of ground motion are listed in Tables 3-1 and 3-3, respectively. The uncertainties in the level of a ground motion parameter given a certain

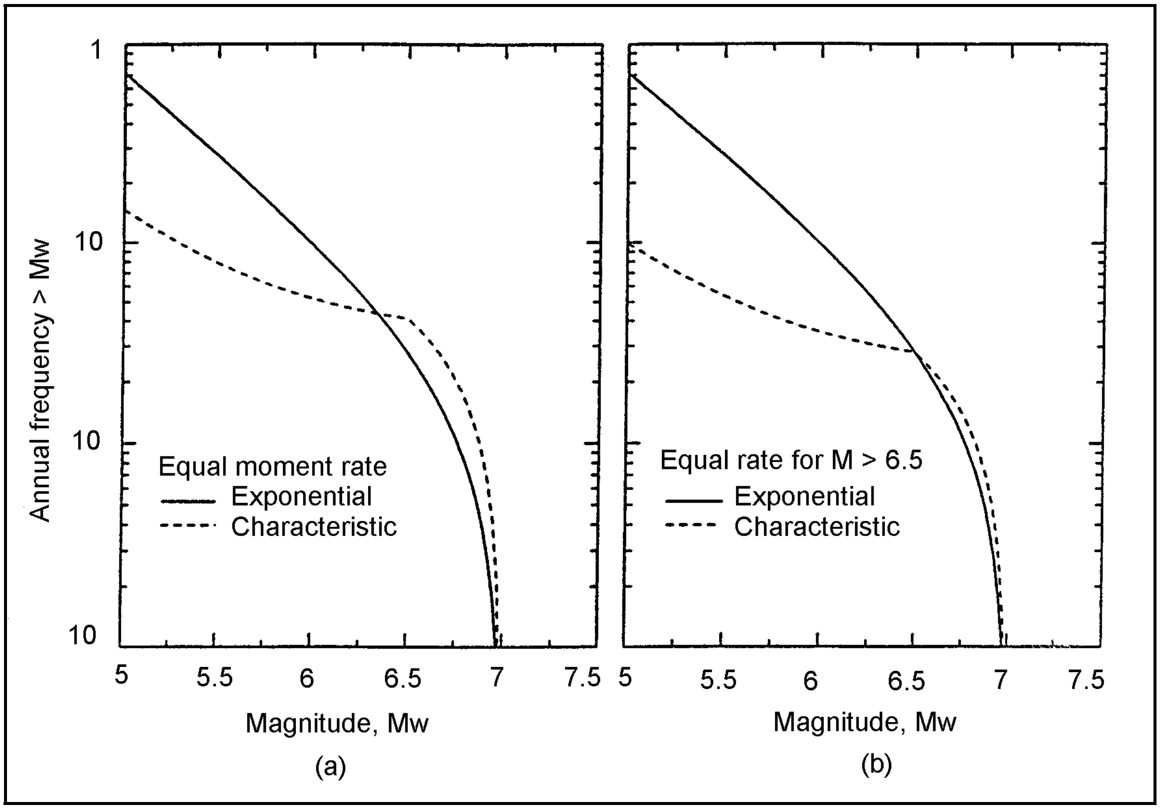


Figure 3-28. Comparison of truncated exponential and characteristic earthquake recurrence relationships

earthquake magnitude and distance (modeled by the probability distribution for the attenuation relationship (Figure 3-26)) are of considerable importance in influencing the results of a PSHA and should be included in the analysis.

3-11. Treatment of Scientific Uncertainty in PSHA

The basic probability formulations in Equations 3-1 and 3-2 incorporate the randomness of the physical process of earthquake generation and seismic wave propagation. Although these formulations incorporate the inherent uncertainty due to randomness, they do not incorporate additional sources of uncertainty that may be associated with the choice of particular models or model parameters. For example, there could be uncertainty about which ground motion attenuation relationship is most applicable to a site, whether an exponential or characteristic earthquake recurrence model is most applicable, the most appropriate model for seismic source zones, the geometry of earthquake sources, the values of maximum earthquake magnitude, or earthquake recurrence parameters. In a deterministic analysis, these uncertainties, which are termed scientific or epistemic uncertainties, are usually treated by applying conservatism in selecting design earthquakes and estimating ground motions. In PSHA, these uncertainties can be directly modeled within the analysis framework to provide an assessment of the uncertainty in the result. The technique of “logic trees” has been widely used to incorporate scientific uncertainty in a PSHA (Kulkarni, Youngs, and Coppersmith 1984; Youngs et al. 1985; Coppersmith and Youngs 1986; National Research Council 1988). Figure 3-29 shows an example of a logic tree used in a PSHA. Although only a few branches of the logic tree are shown, there may be many thousands of branches in the tree. Each path through the tree to an end branch (on the right side of Figure 3-29)

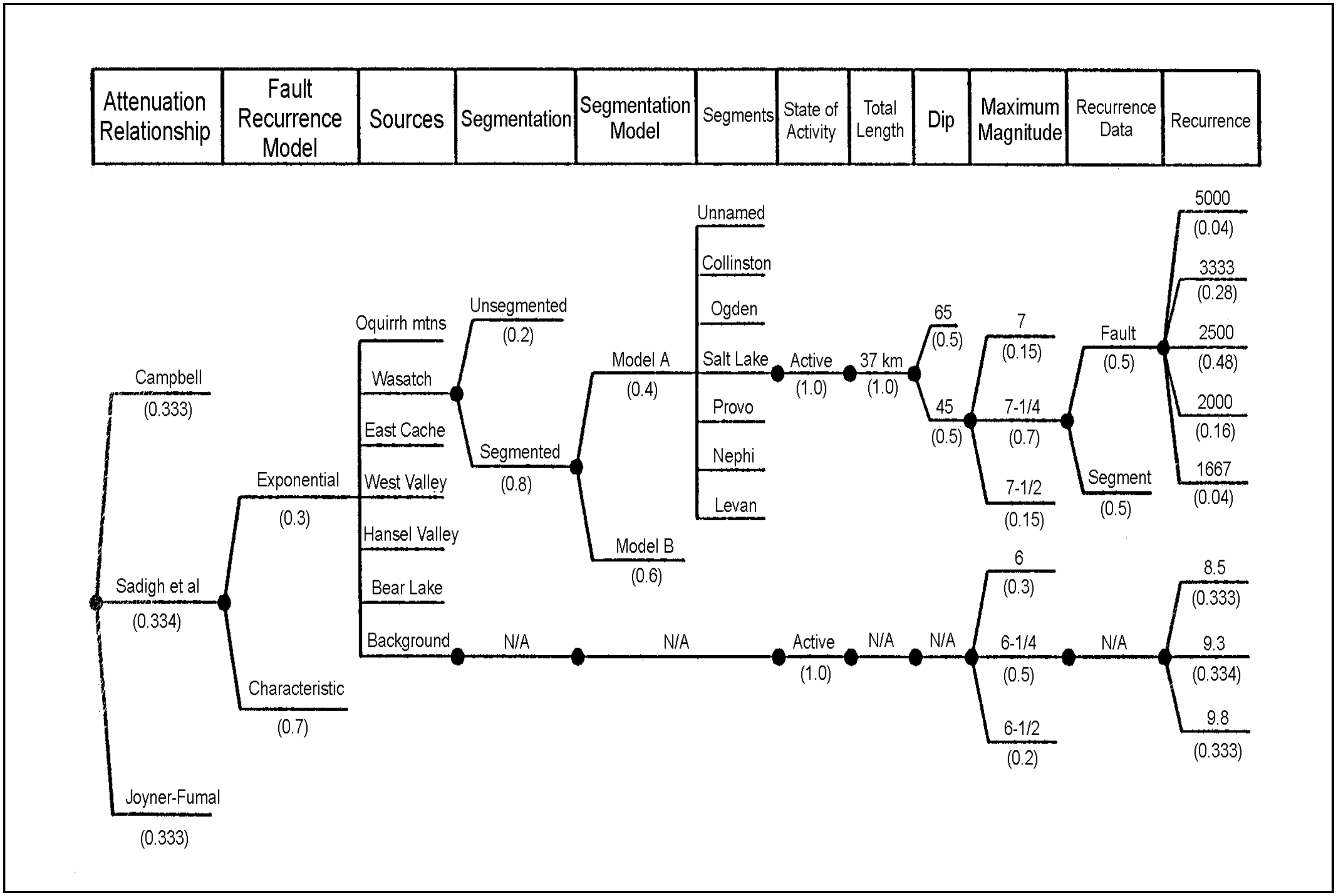


Figure 3-29. Example of logic tree for characterizing uncertainty in seismic hazard input (Youngs, Swan, and Power 1988, reprinted by permission of ASCE)

defines a set of parameters that are used to conduct a basic seismic hazard analysis for that path and end branch using Equation 3-2. Basic hazard analyses are carried out for each path. Each path also has an associated probability or weight that is determined by the product of the relative probabilities or weights assigned to the various models and parameters along the path. (The relative probabilities or weights of the alternative models and parameters are illustrated by the numbers in parentheses in Figure 3-29.) The relative probabilities or weights assigned to alternative models or parameter values are often assigned subjectively, on the basis of the preponderance of scientific evidence or judgment. The sensitivity of the PSHA to changes in the weights can be tested. For some parameters, such as earthquake recurrence based on observed seismicity, the relative weights assigned to different recurrence rates and b-values can be derived from statistical analysis of the seismicity data. The basic hazard analysis results for all the paths in the logic tree are combined using the associated weights to arrive at best estimates (mean or median values) for the frequencies of exceedance of ground motions as well as uncertainty bands for the estimates. Through the approach of incorporating scientific uncertainty, PSHA incorporates the alternative hypotheses and data interpretations that may significantly affect the computed results. The use of logic trees in PSHA, including the mathematical formulation, is discussed in more detail in Appendix F. A simplified example of a PSHA illustrating the calculation process using logic trees is presented in Appendix G (Example G-1).

3-12. Development of Site-Specific Response Spectra from PSHA

The approaches that can be followed in specifying site-specific spectra on the basis of a probabilistic seismic hazard analysis mirror those outlined in Section II for deterministic analyses and involve either anchoring a spectral shape to a peak acceleration level determined from a PSHA for peak acceleration (Figure 3-21) or estimating the entire spectrum on the basis of PSHA for response spectral values at a number of periods of vibration (i.e., developing an equal-hazard spectrum, Figure 3-22). For soil sites, the ground motions can be obtained either by conducting the PSHA using attenuation relationships for soil sites or by conducting the PSHA for rock site conditions and then using site response analyses to evaluate the effects of the site soil column on the ground motions.

a. Approach 1 - Anchoring spectral shape to peak ground acceleration determined from PSHA. In this approach a probabilistic seismic hazard analysis is conducted to establish the relationship between peak ground acceleration and frequency of exceedance. The design peak acceleration level is specified by selecting an appropriate frequency, return period, or probability level. For example, Figure 3-20 shows a typical result of a PSHA for peak ground acceleration. If the exceedance frequency is taken to be 0.001 (return period of 1,000 years), then the corresponding peak acceleration level would be approximately 0.5 g for the example in Figure 3-20. A site-specific spectrum could then be constructed by anchoring an appropriate spectral shape to this acceleration level. As discussed in Section II, it is desirable to select a spectral shape appropriate for the earthquake size and distance producing the hazard as well as the local subsurface conditions. Consideration of earthquake size and distance is less straightforward in a PSHA than in a deterministic analysis because the hazard is the result of the possible occurrences of many different earthquakes of varying sizes and distances from the site. Thus, the hazard analysis results must be examined to identify the major contributors to the hazard at the ground motion level of interest. As an example, Figure 3-30 shows the relative contribution of earthquakes of various magnitudes and distances from three different return periods. As indicated in the figure, the major contribution to seismic hazard shifts to larger magnitudes and closer distances as the return period increases. The example also indicates that a wide range of magnitudes can contribute to the hazard at a selected probability level. This suggests that more than one spectral shape may be appropriate in particular circumstances to address the different types of events that may affect the site.

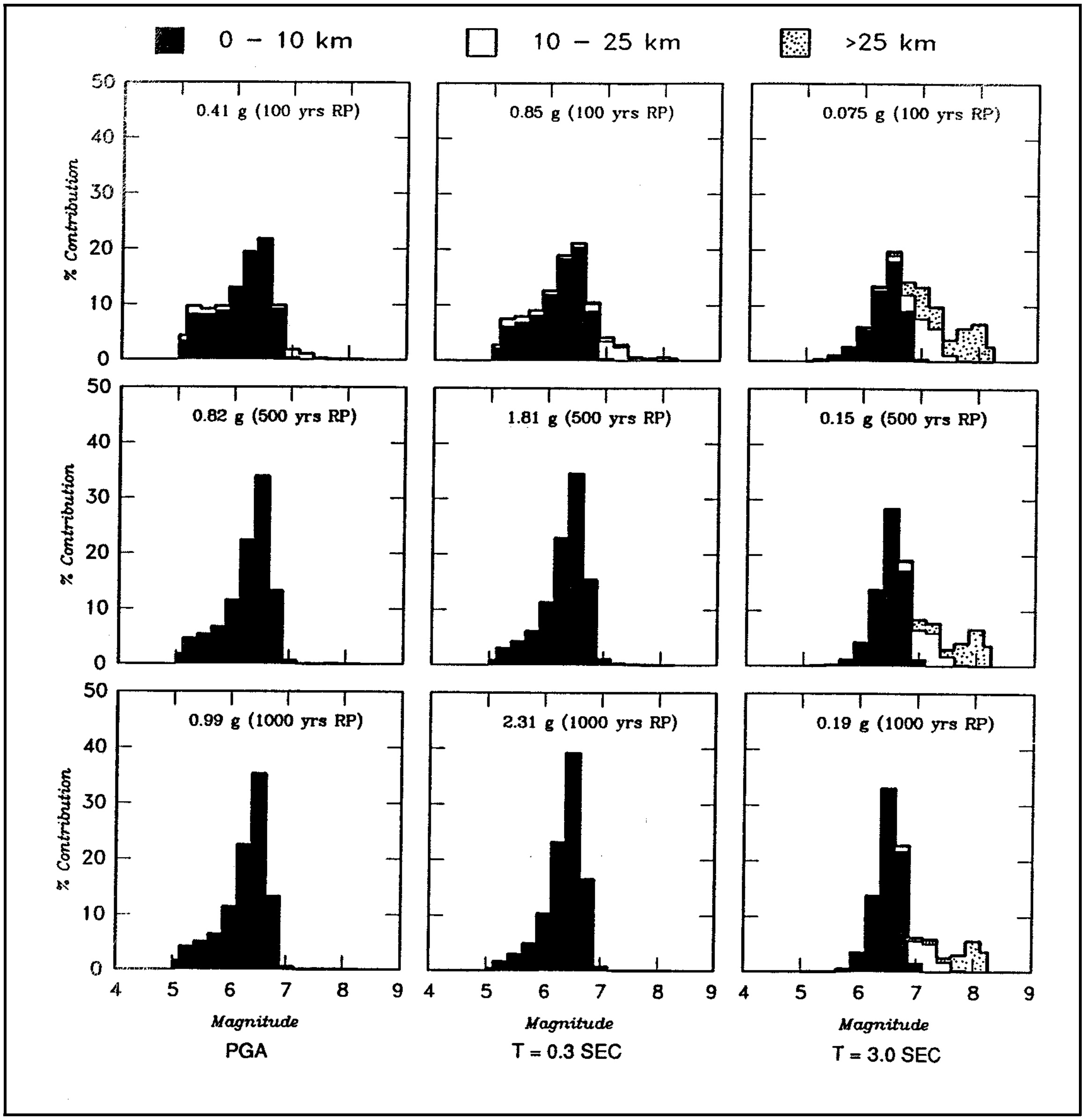


Figure 3-30. Example of contributions of events in various magnitude and distance intervals to mean hazard for peak acceleration and 5 percent damped spectral acceleration at periods of 0.3 and 3 sec

b. Approach 2 - Development of equal-hazard spectra. In this approach, PSHA's are conducted for response spectral values covering the range of vibrational periods of interest for the project. Figure 3-31a shows the results of PSHA's for peak ground acceleration and for 5 percent-damped spectral ordinates at seven selected periods of vibration for the example site used in Figure 3-20. When the appropriate exceedance frequency or return period to use for design is specified, spectral ordinates are read off each hazard curve and are plotted against frequency as shown in Figure 3-31b. (Note that peak ground acceleration is equal to zero-period response spectral acceleration, which, in this example, is equal to response spectral acceleration at 0.03-sec period.) A smooth spectral shape is then drawn through these points to construct the equal-hazard spectrum, a spectrum that has the same probability of exceedance at each frequency. As was the case for the peak ground acceleration hazard results, the

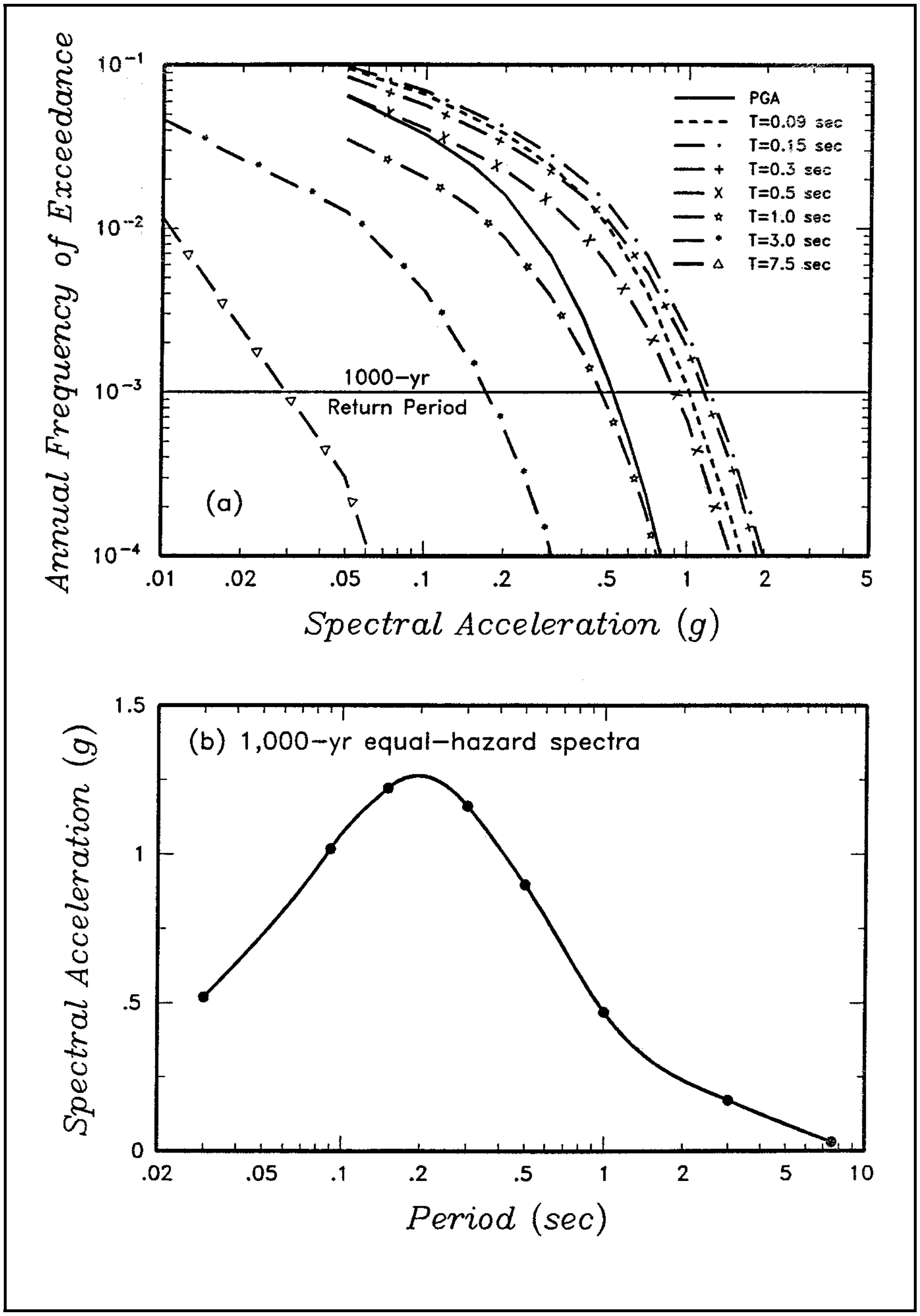


Figure 3-31. Construction of equal-hazard spectra. Top plot (a) shows hazard curves for a range of spectral periods. Bottom plot (b) shows equal hazard spectrum for a period of 1,000 years

equal-hazard spectrum is the result of many possible earthquakes of different sizes and locations. This is illustrated in Figure 3-30, which shows the relative contributions of different magnitude earthquakes to the hazard as a function of return period for spectral values at two periods of vibration as well as for peak ground acceleration. As can be seen, for the same probability of exceedance, the contributions shift to larger magnitudes as the spectral period of vibration increases. This is because the ground motion attenuation relationships are more strongly a function of magnitude at long periods than short periods of vibration.

c. *Preferred approach.* Approach 2 is recommended over Approach 1 because it is not straight-forward to select appropriate spectral shapes in Approach 1. It will be feasible to develop an equal-hazard response spectra from a PSHA using Approach 2. Response spectral value attenuation relation-ships are available for both the EUS and WUS (Section II and Table 3-3) that are as reliable as those developed for peak ground acceleration. The computation of equal-hazard spectra using these spectral attenuation relationships directly accounts for the changes in contributions to the hazard from different magnitudes and source-to-site distances. The use of equal-hazard spectra also accounts for the change in spectral shape with change in return period or probability level, as is illustrated in Figure 3-32 showing an example of equal-hazard response spectral shapes for three probability levels. These are compared in the figure to a standard spectral shape for a similar site condition.

3-13. Development of Accelerograms

Appropriate accelerograms for use with probabilistically based response spectra can be developed using the same two methods described in Section II for deterministic analyses. The additional step required for probabilistically based response spectra is identification of the appropriate magnitude and distance range from which candidate accelerograms may be selected. This information is obtained by deaggregation of the composite hazard to identify the contributions of various magnitudes and distances, as illustrated in Figure 3-30. Suites of natural accelerograms representing the range of events with major contributions to the hazard may be selected and scaled to approximately correspond to the level of the equal-hazard spectrum. Alternatively, synthetically modified accelerograms can be generated to provide a close match to the equal-hazard spectrum (paragraph 3-6c).

3-14. Summary of Strengths and Limitations of DSHA and PSHA

a. *DSHA.*

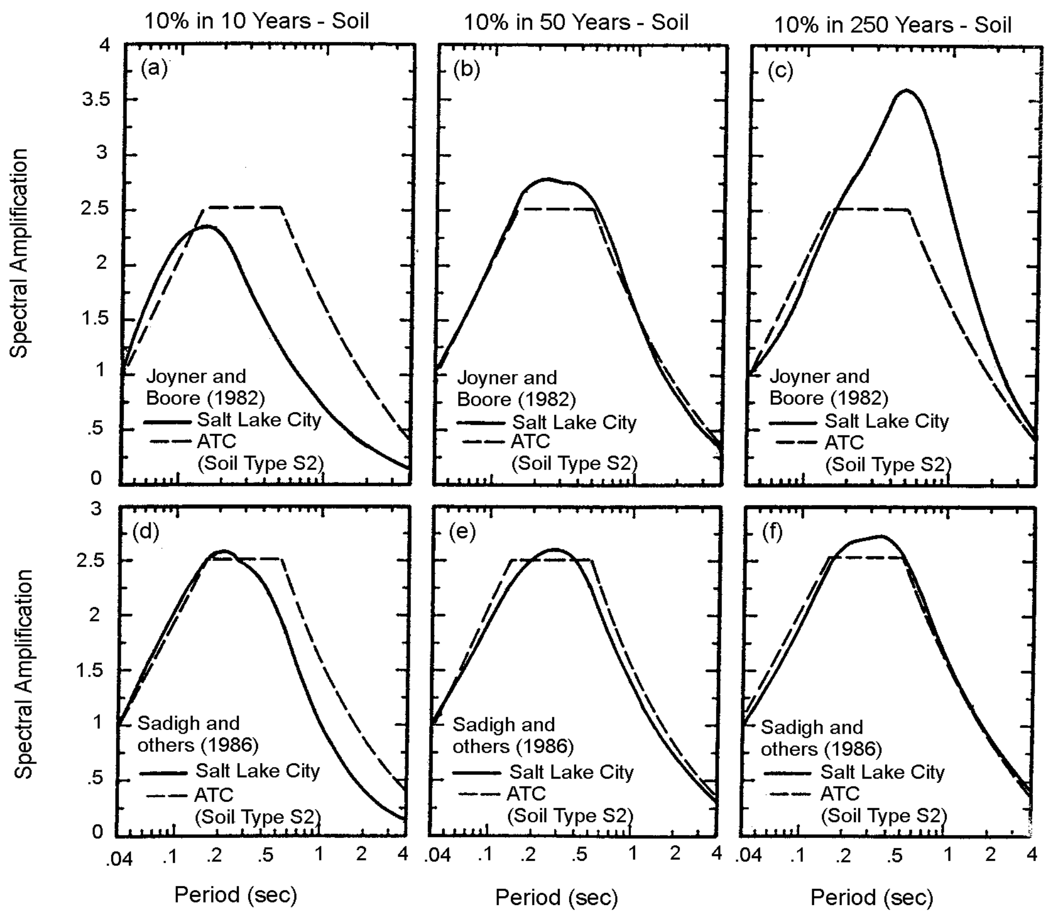
(1) Strengths.

(a) The concept of the design maximum earthquake (MCE) is straightforward and readily understood by the engineer. The MCE represents an estimate of the maximum earthquake size on a source and is located a defined distance from the site.

(b) Provided an appropriate degree of conservatism is incorporated in defining the earthquake magnitude, source-to-site distance, and resulting site ground motions, design for the MCE should provide an appropriately high level of safety.

(2) Limitations.

(a) The frequency of earthquakes and resulting ground motions is not explicitly considered. As a result, a deterministic estimate for a maximum earthquake will have a lower probability of being exceeded in a low-seismicity environment than in a high-seismicity environment.



Notes: 1. Spectral attenuation curves of Joyner and Boore (1982) used to obtain results shown in (a), (b), and (c); spectral attenuation curves of Sadigh, Egan, and Youngs (1986) used to obtain results shown in (d), (e), and (f).

2. Applied Technology Council (1978) (ATC) spectral shape for soil type S2 (deep, firm soil) shown for reference.

Figure-3-32. Response spectral shapes for different probability levels resulting from probabilistic hazard analysis for a deep soil site in Utah

(b) The uncertainties and scientific judgments that are present in a DSHA may not be explicitly recognized or quantified (e.g., uncertainties and judgments in assigning the MCE). As a result, the degree of uncertainty or conservatism in a deterministic estimate is not always known or apparent.

(c) For regions in which active faults have not been identified and sites are located within seismic source zones (e.g., EUS), there is not a standard approach or clear basis for selecting the distance to the design earthquake.

b. PSHA

(1) Strengths.

(a) A PSHA allows the designer to balance risk and cost for a project in a manner similar to that used for other environmental loadings such as flood or wind loadings. The reduction in risk by selecting a lower probability level (longer return period) and correspondingly higher seismic loading may be compared to the increased project cost involved with designing for the higher loading.

(b) The frequency of occurrence of earthquakes is explicitly incorporated in a PSHA. As such, regions of greater seismic activity (thus higher probabilities of earthquake occurrence) will have higher ground motion levels for given probabilities or return periods.

(c) The uncertainty or randomness in earthquake location is explicitly incorporated in a PSHA. Thus, the conservatism of assuming that the earthquake occurs at the closest location on the source to the site is not necessary in a PSHA. For sites located within low-seismicity seismic source zones in the EUS, it is usually very conservative to “float” the earthquake to the site or even within a small radius of the site.

(d) Uncertainties in the earthquake occurrence and ground motion estimation process are explicitly considered in a PSHA.

(2) Limitations.

(a) The fact that there are significant scientific uncertainties in earthquake source characterization and ground motion estimation means that there is not a unique result for the relationship between ground motion level and probability of exceedance. In fact, there is usually a significant range of possible ground motion levels for a given probability of exceedance. Usually, the mean estimate from a logic tree analysis is used as a basis for selecting project design criteria.

(b) Probability of exceedance versus ground motion relationships and equal hazard spectra from PSHA's involve contributions from multiple earthquake sources, magnitudes, and distances. As a result, the concept of a design earthquake is not as straightforward as in a DSHA. Furthermore, the evaluation of the duration of shaking and selection of acceleration time-histories based on probabilistic results are more difficult than for a deterministic analysis because of the multiple contributions. Therefore, it is important to deaggragate the results of a PSHA so that the primary contributors (earthquake sources, magnitude ranges, distance ranges) are known.

(c) While uncertainties are incorporated in the analysis (e.g., by the logic tree approach), nevertheless the weights assigned to alternative models and parameter values generally involve subjective judgment. The basis for these judgments should be fully documented.

c. Summary. Both deterministic and probabilistic ground motion analyses have their place in developing earthquake ground motions for seismic design. These two approaches should be used to complement each other as specified in ER 1110-2-1806. It is usually appropriate to carry out both types of analyses to aid the design engineer in developing the project seismic design criteria. Of paramount importance in either deterministic or probabilistic analyses is the expertise of the individuals conducting the analyses.

3-15. Examples of PSHA

Appendix G presents examples of PSHA. The first example is a simplified calculation that illustrates the basic calculational process and probability functions. The other examples represent actual applications in the WUS and EUS. These examples also illustrate how deterministic and probabilistic analyses can be used together in selecting project seismic design ground motions.