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PROCEDURES FOR EVALUATION OF VIBRATORY GROUND MOTIONS OF SOIL DEPOSITS AT NUCLEAR POWER PLANT SITES

Prepared Under
Contract No. AT(04-3)-954

U. S. NUCLEAR REGULATORY COMMISSION
OFFICE OF NUCLEAR REGULATORY RESEARCH
WASHINGTON, D.C.

JUNE 1975

SW-AA

A Joint Venture of
SHANNON & WILSON, INC. AND AGBABIAN ASSOCIATES
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ABSTRACT

According to USNRC requirements set forth in Appendix A, 10 CFR, Part 100, vibratory ground motion criteria for a nuclear plant must be based on local soil conditions, as well as on the seismicity, geology, and tectonics of the region. This report describes how such criteria can be developed by applying the latest technology associated with analytical predictions of site-dependent ground motions and with the use of composite spectra obtained from the current library of strong motion records. The report is organized around a recommended procedure for defining vibratory ground motion criteria; this procedure contains the following steps: (1) geologic and seismologic studies; (2) site soils investigations; (3) site response sensitivity studies; (4) evaluation of local site response characteristics; (5) selection of site-matched records; and (6) appraisal and selection of seismic input criteria.

To provide a suitable base from which to implement the above procedure, the report is divided into four main chapters. The first contains an in-depth discussion of the engineering characteristics of earthquake ground motions. This includes parameters used to characterize earthquakes and strong motion records, geologic factors that influence ground shaking, the current strong motion data base, and case histories of the effects of past earthquake events. Next, geotechnical investigations of the seismologic, geologic, and site soil conditions required to develop vibratory motion criteria are briefly summarized. The following chapter describes the current technology for establishing vibratory ground motion criteria at nuclear plant sites, including site-independent and site-dependent procedures that use data from strong motion records and from soil response analyses. The final chapter contains a detailed description of the recommended procedure and provides an example application.

FOREWORD

The Nuclear Regulatory Commission (USNRC) has undertaken a program to improve understanding of soil behavior under seismic loading conditions. As a part of this program, the firms of Shannon & Wilson, Inc. and Agbabian Associates (SW-AA) have been retained by the USNRC as a joint venture under Contract No. AT(04-3)-954 to conduct research that is directed toward reviewing present concepts for evaluating the dynamic response of site soil deposits; recommending an orderly system of field investigations, laboratory studies, and analyses sufficient to define the pertinent physical properties of the subsurface soils; and providing specific recommendations for quantitative assessment of earthquake-induced ground response for prospective reactor sites. This report is part of the overall research effort.

The purpose of this report is to indicate how local soil conditions can be considered in the development of seismic input criteria that are consistent with USNRC criteria requirements set forth in Appendix A, 10 CFR, Part 100. However, this report does not represent a USNRC position with respect to methodology or to procedures for compliance with the criteria. The report contains an in-depth discussion of background information and current technology for establishing earthquake ground motion criteria using site-independent and site-dependent techniques. In addition, geotechnical investigations required as a basis for establishing such criteria are briefly summarized. A recommended procedure for determining seismic input criteria for nuclear plants is then provided and is illustrated with an example application.

The project manager for the SW-AA joint venture effort is R. P. Miller of Shannon & Wilson, Inc. Project engineer for Agbabian Associates and principal investigator for the preparation of this report is S. D. Werner. Significant contributions to the report were made by M. S. Agbabian, S. D. Werner, J. W. Workman, and G. A. Young of Agbabian Associates; and I. Arango, R. P. Miller, H. H. Waldron, and S. D. Wilson

of Shannon & Wilson, Inc. Dr. H. B. Seed of the University of California at Berkeley served as special consultant to the joint venture on this report.

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CHAPTER 1

DESCRIPTION OF REPORT

Recent trends in the electrical power industry indicate that during the next few years, a substantial expansion in the construction of nuclear power plants will take place to meet increasing requirements for power. Because nuclear plants present unique problems with regard to public safety, such facilities must meet exacting specifications for plant siting, design, construction, operation, and containment of the generated energy and radiation. The Nuclear Regulatory Commission (USNRC), formerly the Atomic Energy Commission (AEC), is charged with the responsibility for regulating all uses of nuclear energy. To fulfill this function, the USNRC establishes design requirements and reviews each license application to ensure that these requirements have been met. In addition, the USNRC continuously evaluates engineering, construction, and operational safety procedures, updating them as new advances in the technology for describing environmental hazards and system responses are developed.

In order to provide adequate operation, and energy control and containment, many factors rarely considered in the design of conventional structures must be carefully evaluated when designing nuclear plants. Of these factors, the resistance of a nuclear power plant to the effects of strong motion earthquakes is one of the more significant conditions to be considered by the electrical power generating industry.

Earthquakes present a unique hazard because their potential loads on nuclear plants are among the largest that can be applied by natural causes. In addition, the ground shaking induced by earthquakes may cause landslides, subsidence, or liquefaction of the soil mass in and around the plant site. As a result, the USNRC and their consultants and contractors have been deeply involved in the development of improved techniques for estimating potential earthquake ground motions at a particular site and in the evaluation of the effects of these ground motions on the site soil materials and on nuclear

plant structures, equipment, and piping. This report, which deals specifically with techniques for incorporating local site conditions in the selection of seismic input criteria for nuclear plants, is prepared as part of this overall development effort.

1.1 PURPOSE AND SCOPE

To provide a basis for site selection and the establishment of design requirements arising from seismic and geologic considerations, an appendix to the USNRC criteria, 10 CFR, Part 100, entitled "Appendix A - Seismic and Geologic Siting Criteria for Nuclear Power Plants," has been formulated (Ref. 1-1). This appendix, which is to be followed by all prospective applicants for nuclear plant operating licenses, contains seismological and geotechnical requirements and procedures for determining seismic input criteria at a site. This includes a consideration of such potential earthquake-induced effects as vibratory ground motions, soil and slope instabilities, floods, and water waves. These criteria are to be incorporated in preliminary and final safety analysis reports, prepared according to standard USNRC format and content requirements (Ref. 1-2). In addition, the criteria must be used in conjunction with USNRC engineering requirements for the design of the nuclear plant to accommodate the vibratory effects of earthquakes (Ref. 1-3).

The procedures specified in Appendix A, 10 CFR, Part 100 for the establishment of earthquake ground motions at a site, provide for investigations of the seismicity, geology, tectonics, and local soil conditions at the site. In particular, requirements for consideration of local soil effects are clearly specified in Section IV, Paragraph (a-4) in which it is stated that the soil properties determined at the site should be "properties needed to determine the behavior of the underlying material during earthquakes and

the characteristics of the underlying material in transmitting earthquake-induced motions to the foundations of the plant." Furthermore, in Section V, Paragraph (a-1-iv), it is specified that

The maximum vibratory accelerations of the Safe Shutdown Earthquake at each of the various foundation locations of the nuclear power plant structures at a given site shall be determined taking into account the characteristics of the underlying soil material in transmitting the earthquake induced motions....

The purpose of this report is to describe how local soil conditions can be considered in the development of seismic input criteria consistent with the above requirements of Appendix A, 10 CFR, Part 100. The report contains the background information and evaluations of current technology required to provide a basis for sound engineering decisions regarding input criteria determination. Only vibratory ground motion requirements are included, and consideration of those engineering design requirements in the USNRC criteria not specifically directed toward vibratory ground motions is beyond the scope of this report.*

1.2 REPORT ORGANIZATION

The organization of the report is closely correlated with a recommended procedure for the determination of seismic input criteria for a nuclear plant site, as shown in Figure 1-1. As indicated in this figure, the procedure provides for investigations of seismic, geologic and local soil conditions; evaluations of local soil response characteristics based on a series

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*Among the engineering design requirements included in Appendix A, 10 CFR, Part 100 that are not specifically directed toward vibratory ground motions are evaluations of the hydrology of the area, the potential for and effects of surface faulting at the site, soil liquefaction, slope instabilities, earthquake-induced floods, tsunamis, etc.

FIGURE 1-1. REPORT ORGANIZATION AND PROCEDURE FOR SELECTION OF SEISMIC INPUT CRITERIA FOR NUCLEAR PLANT SITE

of site response sensitivity studies; and selection of an ensemble of site-matched ground motion records that best represent the particular seismologic, geologic, and local soil conditions at the potential nuclear plant site. The results from each of these steps are considered in conjunction with statistically determined spectrum shapes provided in USNRC Regulatory Guide 1.60 (Ref. 1-4) in order to determine the seismic input criteria for the site.

To provide information for implementing each of the six steps shown in Figure 1-1, the remainder of this report is divided into four chapters. Chapter 2 provides a description of the current state of knowledge associated with the characterization of earthquake ground motions. Chapter 3 briefly summarizes the geologic, seismologic, and site-soils investigations required to establish the significant earthquakes to be considered and the soil properties required for site response analyses (Steps 1 and 2 of Fig. 1-1). Chapter 4 describes site-independent and site-dependent techniques for predicting earthquake ground motions based on existing strong motion records and on soil response analyses. This information should be used with Steps 3 through 6 of the procedure shown in Figure 1-1. Finally, Chapter 5 summarizes the overall six-step procedure and contains an example illustrating how the procedure can be applied to an actual site.

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- 1-1. U.S. Atomic Energy Commission, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," Appendix A, 10 CFR, Part 100, *Federal Register*, Vol. 38, No. 218 (13 November 1973).
- 1-2. *Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants*. Washington, D.C.: U.S. Atomic Energy Commission, Regulatory Staff, February 1972.
- 1-3. "Seismic Requirements for Design of Nuclear Power Plants and Test Facilities," *RDT Standard*, RDT F9-2T. Washington, D.C.: U.S. Atomic Energy Commission, Division of Reactor Research and Development, January 1974.
- 1-4. U.S. Atomic Energy Commission, *Design Response Spectra for Seismic Design of Nuclear Power Plants*, Regulatory Guide 1.60. Washington, D.C.: Directorate of Regulatory Standards, October 1973.

CHAPTER 2

ENGINEERING CHARACTERISTICS OF EARTHQUAKE GROUND MOTIONS

The purpose of this chapter is to summarize the general properties of earthquake ground motions that must be considered when specifying seismic input criteria at a nuclear plant site. The chapter contains five main sections. The first discusses various techniques and parameters used by engineers to characterize strong motion earthquakes and records of ground shaking. Next, the geologic factors that influence the characteristics of the ground shaking at a site are described. The third section contains a description of the strong motion data base from which the engineer must draw when formulating the seismic input criteria. The fourth section describes several case histories of actual earthquakes during which damage or strong motion measurements have had varying degrees of dependence on local soil vibratory effects. The final section summarizes this information and relates it to the determination of seismic input criteria for nuclear power plants.

2.1 CHARACTERIZATION OF STRONG MOTION EARTHQUAKES

In this section, various parameters used by engineers to characterize and classify strong motion earthquakes are described. Included are the earthquake epicenter and focus, magnitude, intensity, and various parameters for classifying strong motion strength levels.

2.1.1 EPICENTER AND FOCUS OF AN EARTHQUAKE

Earthquakes generally occur when the strength of the rock can no longer withstand elastic strain that has built up because of slippage along an active fault. The location along the fault where this strain is released is characterized by the epicenter and focus of the earthquake.

The focus or hypocenter of the earthquake is the point within the earth's crust where the initial rupture of the rocks occurs and the elastic waves from the earthquake are first released. The majority of earthquakes recorded in the United States have had shallow focal depths (10 to 15 km) and have occurred in regions containing surface fault patterns (such as California, for example). In other regions, however, such as the Puget Sound area and many parts of the eastern United States, earthquakes are focused at deeper locations within the earth's crust so that a surface rupture is not observable in the field. In the Puget Sound area, for example, focal depths are typically about 40 km (Refs. 2-1, 2-2). This is still relatively shallow when compared to the focal depths of 500 to 700 km that have been reported in other parts of the world (Ref. 2-3).

The epicenter of the earthquake is the vertical projection of the focus onto the earth's surface. In the absence of instrumental data, the epicenter has been established on the basis of damage observed. However, epicenters are now typically located based on relative arrival times of seismic wave components at various instruments within a seismograph network. Neither epicenter nor focus location procedures are precise, due to limitations in the number of reliable recording stations as well as in the geologic interpretations of crustal structures and knowledge of local propagation velocities in many areas.

It is important to recognize that the epicenter and hypocenter do not necessarily indicate the center of total energy release of the earthquake -- rather, they indicate the point where the fault slip began. For smaller earthquakes, the center of this energy release and the point where slip began are not far apart; however, for large earthquakes, these points may be hundreds of miles apart. For example, the 1960 Chilean earthquake had a slipped fault length of about 600 mi and an epicenter at the northern end of the slip, some 300 mi from the center of energy release (Refs. 2-4, 2-5).

2.1.2 MAGNITUDE

The magnitude of an earthquake is intended to be a measure of its size, independent of the place of observation. It is calculated from measurements on seismographs and is therefore expressed in ordinary numbers and decimals. Physically, the magnitude has been correlated with the energy released by an earthquake, as well as with the fault rupture length and the maximum fault displacement (Refs. 2-1, 2-3).

At present, at least three different magnitudes are in common use for classifying earthquakes: (1) local magnitude, based on peak response of a defined instrument; (2) body-wave magnitude, based on the response amplitude of a particular body-wave type; and (3) surface-wave magnitude, based on the response amplitudes of long-period seismic waves (Ref. 2-6). Each of these magnitudes is derived from a well-calibrated instrument, some knowledge of the characteristics of the rock through which the seismic waves must travel, and the local conditions at the instrument station.

The local magnitude, defined by Richter in 1935, is perhaps the most widely used magnitude measure. This magnitude, M , is defined as (Ref. 2-3):

$$M = \log_{10} \frac{A}{A_0} \quad (2-1)$$

where

A = Maximum trace amplitude, in microns, recorded on a standard Wood-Anderson seismograph located on firm ground 100 km from the epicenter

A_0 = Reference trace amplitude (taken as 1 micron at a distance of 100 km from the epicenter)

In practice, empirical charts are used to correct for epicentral distances other than 100 km and for various site conditions at a given seismograph station.

It is noted from Equation 2-1 that each upward step of one magnitude unit corresponds to a ten-fold increase in the recorded amplitude, A . Furthermore, Richter has shown that for shallow earthquakes in California, the magnitude is related to the energy released by an earthquake, E , according to the following expression:

$$\log_{10} E = 11.4 + 1.5 M \quad (2-2)$$

Therefore, the energy associated with an earthquake increases by more than an order of magnitude with each unit increase in magnitude (Ref. 2-3).

Although the magnitude is a convenient way of classifying earthquakes, its limitations should be recognized. First, owing to the nonuniformity of the earth's crust, to the possible effects of the orientation of the fault relative to the seismograph stations, and to uncertainties in the corrections for seismograph site conditions, the magnitude is not a precise measure of the size of an earthquake. Second, the magnitude of an earthquake is not a good indicator of the peak ground acceleration, since the magnitude is a measure of the amplitudes of long period waves at 100 km from the source, whereas peak accelerations are related to higher frequency motions measured at any location relative to the source.

2.1.3 INTENSITY

In the absence of instrumental recordings of ground motion, seismologists have described the shaking by assigning intensity numbers according to various subjective intensity scales. To describe earthquakes that occurred prior to the early 1900's, the Rossi-Forel scale containing 10 grades of intensity was typically utilized. However, with subsequent advances in

technology, the Rossi-Forel scale grew out of date, primarily because of the lumping of an enormous range of intensity at its highest level, X, and because of the heavy European orientation of its scale descriptions. As a result, a new improved scale was developed by Mercalli in 1902. This scale was originally developed as a 10-step scale and was later expanded to a 12-step scale following a suggestion by Cancani. Elaborations of the 12-step scale were published by Sieberg in 1923. The Modified Mercalli (MM) intensity scale, as used today in the United States, was developed by Wood and Newman in 1931 (Ref. 2-3).

The Rossi-Forel and MM intensity scales are compared in Figure 2-1a (Ref. 2-7). Other scales currently applied throughout the world include the Cancani-Sieberg (1917) version of the original Mercalli scale, used extensively in western Europe, and the Medvedev, Sponheuer, and Karnik (1964) scale. All of these scales are subjective and are similar in structure to the MM intensity scale (Refs. 2-2, 2-3).

Following an earthquake, the assignment of an intensity to a given location is based on interviews with inhabitants of the area and on observations of damage in the area. For earthquakes occurring in earlier historic times, newspaper accounts and passages from journals, books, and other contemporaneous material are used to assign intensities. Assigned intensity values at different locations are then combined to formulate a map containing a series of isoseismals, or curves that separate regions of successive intensity ratings. These maps, termed isoseismal or intensity maps, reflect the attenuation of damage with distance from the source, and the extent of the felt area of the earthquake. The shapes and extent of the isoseismals may be influenced by the tectonic features of the area, indicating predominant directions along which seismic waves are transmitted and the manner in which the earthquake originates. An intensity map for the 1971 San Fernando earthquake is shown in Figure 2-1b (Ref. 2-8).

ROSSI-FOREL INTENSITY SCALE (1883)	MODIFIED-MERCALLI INTENSITY SCALE (1931)
	I Detected only by sensitive instruments.
I The shock felt only by experienced observer under very favorable conditions.	II Felt by a few persons at rest, especially on upper floors; delicate suspended objects may swing.
II Felt by a few people at rest, recorded by several seismographs.	III Felt indoors, but not always recognized as a quake; standing autos rock slightly; vibration similar to passing truck.
III Felt by several people at rest; strong enough for the duration or direction to be appreciable.	IV Felt indoors by many, outdoors by a few; at night some awaken; dishes, windows, doors disturbed; motor cars rock noticeably.
IV Felt by several people in motion; disturbance of movable objects, cracking of floors.	V Felt by most people; some breakage of dishes, windows, and plaster; disturbance of all objects.
V Felt generally by everyone; disturbance of furniture, ringing of some bells.	VI Felt by all; many frightened and run outdoors; falling plaster and chimneys; damage small.
VI General awakening of those asleep, ringing of bells, swinging chandeliers, startled people run outdoors.	VII Everybody runs outdoors; damage to buildings varies, depending on quality of construction; noticed by drivers of autos.
VII Overthrow of movable objects, fall of plaster, ringing of bells, panic with great danger to buildings.	VIII Panel walls thrown out of frames; fall of walls, monuments, chimneys; sand and mud ejected; drivers of autos disturbed.
VIII Fall of chimneys; cracks in walls of buildings.	IX Buildings shifted off foundations, cracked, thrown out of plumb; ground cracked; underground pipes broken.
IX Partial or total destruction of some buildings.	X Most masonry and frame structures destroyed; ground cracked; rails bent; landslides.
X Great disasters, ruins; disturbance of strata, fissures, rockfalls, landslides; etc.	XI New structures remain standing; bridges destroyed; fissures in ground; pipes broken; landslides; rails bent.
	XII Damage total; waves seen on ground surface; lines of sight and level distorted; objects thrown up into air.

(a) COMPARISON OF ROSSI-FOREL AND MODIFIED MERCALLI INTENSITY SCALES (REF. 2-7)

(b) MODIFIED MERCALLI INTENSITY MAP--1971 SAN FERNANDO EARTHQUAKE (REF. 2-8)

FIGURE 2-1. INTENSITY SCALES AND MAPS

Subjective intensity scales are an important consideration in those areas where no strong motion records exist, particularly where they afford the only means for interpreting historical information. However, a number of undesirable shortcomings are involved in the utilization of intensities. First, the use of human reactions to assign an intensity scale is strongly dependent on many factors, including the prior exposure of that person to ground motions, his emotional makeup, etc. Second, structural damage is strongly dependent on design and construction practices in a given area. For example, it is especially difficult to evaluate intensities deduced from earthquakes in underdeveloped areas, when compared to those from earthquakes in developed regions. Finally, earthquakes occurring in sparsely populated areas have been assigned intensities based largely on permanent ground displacements or slope failures that, by themselves, are unreliable measures of earthquake damage.

Current design practice often uses interrelationships between MM intensities and quantitative measures of the severity of ground shaking, such as peak acceleration. A number of these intensity/acceleration correlations are shown in Figure 2-2 (Refs. 2-9 to 2-13). The scatter of these various correlations is still another indication of the lack of reliability of the use of intensities for design purposes.

2.1.4 STRONG MOTION PARAMETERS

The best means for depicting earthquake ground motions is the acceleration time history, which expresses the total history of shaking of the ground at the instrument site. However, for purposes of quantitatively comparing different records and for relating measured ground motions to potential structure damage, simple parameters are often used to define a given record. A number of these parameters -- namely, peak acceleration, velocity, or displacement; response spectra and spectrum intensity; root-mean-square acceleration; and duration of strong shaking -- are discussed below.

FIGURE 2-2. EARTHQUAKE HORIZONTAL ACCELERATION VERSUS INTENSITY CORRELATIONS (REFS. 2-9 TO 2-13)

2.1.4.1 Peak Acceleration, Velocity, or Displacement

The peak ground acceleration is presently the most widely used measure of the strength of ground shaking, and most of the relatively scarce data regarding strength levels of past earthquakes is provided in terms of peak accelerations. However, the use of this parameter to represent an earthquake record has a number of limitations. First, the peak acceleration is closely related to lateral forces for high-frequency systems, but not for more typical low- or intermediate-frequency structures. Also, the peak acceleration alone does not lead to good correlations between the relative strengths of different earthquake records, nor does it account for the duration or the frequency content of the motion (Ref. 2-5).

The strength of earthquake-induced ground shaking can also be depicted by peak velocity or displacement measurements, both of which appear to have a more determinant upper bound than accelerations. In addition, the peak displacement and velocity provide an improved estimate of the effective lateral forces applied by earthquakes to low- and intermediate-frequency structures. Of the two, the peak velocity is generally preferred, as indicated by past studies of earthquake and nuclear shock records (Refs. 2-6, 2-14). However, both parameters are subject to errors arising from the numerical integration of the acceleration records; and like the peak acceleration, they do not account for the duration or frequency content of the ground motions.

2.1.4.2 Response Spectra and Spectrum Intensity

In current earthquake engineering practice, the response spectrum is a widely used method for characterizing earthquake ground motions. A response spectrum is a plot of the maximum response of a simple damped structure as a function of its natural period or frequency. A family of response spectra is typically provided, each spectrum corresponding to a given damping ratio. The construction of response spectra for an earthquake record is depicted in Figure 2-3.

NOTE:

$$\text{PSEUDOVELOCITY } \dot{u} = u \frac{2\pi}{T}$$

u = MAXIMUM DISPLACEMENT
OF STRUCTURE
RELATIVE TO BASE

POINT ON SPECTRUM	(A)	(B)	(C)
NATURAL PERIOD	$T = 0.3 \text{ SEC}$	$T = 0.5 \text{ SEC}$	$T = 1.0 \text{ SEC}$
DAMPING RATIO	$\lambda = 0.05$	$\lambda = 0.05$	$\lambda = 0.05$
PSEUDOVELOCITY	$\dot{u} = 13.0 \text{ IPS}$	$\dot{u} = 25.3 \text{ IPS}$	$\dot{u} = 31.6 \text{ IPS}$

(a) RESPONSE OF SIMPLE STRUCTURES

(b) ACCELEROGRAM (= BASE MOTION TO SIMPLE STRUCTURES)

(c) TRIPARTITE SPECTRA

(d) ARITHMETIC SPECTRA -- RELATIVE VELOCITY VS. PERIOD

FIGURE 2-3. CONSTRUCTION OF RESPONSE SPECTRA FOR 1940 EL CENTRO EARTHQUAKE, NORTH-SOUTH COMPONENT OF MOTION

The use of response spectra in seismic design practice is motivated by the ability of these spectra to reveal the effects of the ground motions on the response of a simple structure. Moreover, even for complex structures, response spectra are used in conjunction with approximate techniques for combining peak modal responses. An additional advantage of the use of response spectra as a strong motion parameter is its inclusion of the frequency content of the ground motions.

A disadvantage of response spectra is that they, by themselves, do not provide a single numerical measure of the relative strengths of different earthquake ground motion records. To circumvent this problem, the spectrum intensity of the earthquake record can be used. As originally conceived by Housner, this parameter is defined as the area under a pseudovelocity spectrum of a record between periods of 0.1 to 2.5 sec (Ref. 2-15). The spectrum intensities of 20% damped spectra have been closely correlated with root-mean-square (rms) accelerations of a number of records, although this damping level is not representative of typical structures. The primary advantage of the spectrum intensity is that it correlates the earthquake strength with the peak response of an oscillator in a period range common to many structures. However, this quantity does not provide information regarding the duration of strong shaking.

2.1.4.3 Root-Mean-Square Acceleration

The root-mean-square (rms) acceleration of an earthquake record is defined as

$$rms = \left[\frac{1}{t_d} \int_0^{t_d} \ddot{y}^2(t) dt \right]^{1/2} \quad (2-3)$$

where $\ddot{y}(t)$ is the acceleration at time t of the earthquake record of duration t_d . The prime advantage of the rms acceleration as a measure of the strength of the ground shaking is that it is easily calculated and is applicable to both deterministic and nondeterministic earthquake response analyses (Ref. 2-5).

Arias has proposed a means of classifying the strength of a given record in terms of an intensity factor, I , where

$$I = \frac{\pi}{2g} \int_0^{t_d} \ddot{y}(t)^2 dt \quad (2-4)$$

and g is the acceleration of gravity. A comparison of Equations 2-3 and 2-4 indicates a close correlation between the rms acceleration and Arias' intensity definition. Arias has shown that Equation 2-4 corresponds to the total energy stored at the end of the earthquake in a family of linear undamped oscillators of varying frequency. It is nearly independent of structural damping over a wide range of damping levels (Ref. 2-16).

2.1.4.4 Duration of Strong Shaking

From observations of a number of strong motion records, it has been noted that the time-dependent energy and vibration level variations consist of: (1) an initial segment in which the energy and vibration levels rapidly increase to high values; (2) a segment of uniformly strong shaking at these high levels; and (3) a period of gradually attenuating vibration (Ref. 2-17). The duration of strong shaking, item 2 above, is of particular importance regarding structure damage potential. This strong motion duration is closely related to the energy released by the earthquake which, in turn, is dependent on the length and displacement of the slipped region of the causative fault and on the magnitude of the stress drop that occurred. As indicated in Section 2.1.2, a partial measure of these latter parameters is provided by

the Richter Magnitude. The effect of the magnitude on the duration of strong shaking near the causative fault has been studied by Housner (Ref. 2-18). In addition, recent studies by Bolt (Ref. 2-19) and Page, et al. (Ref. 2-20) have indicated that the duration of strong shaking may also be dependent on the frequency content of the seismic waves and the distance to the causative fault. These interrelationships are discussed further in Section 2.3.

An indication of the importance of the duration of strong shaking is indicated in Figure 2-4. In this figure, the dynamic characteristics of an accelerogram recorded near the epicenter of the 1966 Parkfield earthquake are compared to those of an accelerogram recorded during the 1940 El Centro earthquake. Although the Parkfield record obviously has a much higher peak acceleration and spectrum intensity, the El Centro earthquake caused considerably more damage to structures in the area. This apparent paradox is resolved by considering differences in the duration of strong shaking of the two records; for the Parkfield record this duration is about 1 sec, as opposed to 15 sec for the El Centro record. From this and other similar occurrences, it is seen that earthquake excitations with strong motion over short durations are not as damaging as might be inferred from their peak accelerations or spectrum intensities (Ref. 2-21).

2.2 GEOLOGIC FACTORS AFFECTING GROUND SHAKING AT A SITE

In order to properly apply the current technology for estimating earthquake motions, the various factors that affect the ground shaking at a site must be recognized. These factors are discussed in this section and, as indicated in Figure 2-5 (Ref. 2-6), include the source mechanism, the source-to-site transmission path, and the local site conditions.

(a) N65E GROUND ACCELERATION, PARKFIELD, 1966

(b) N-S GROUND ACCELERATION, EL CENTRO, 1940

(c) COMPARISON OF VELOCITY SPECTRA
(DAMPING RATIO = 0.05)

FIGURE 2-4. COMPARISON OF PARKFIELD AND EL CENTRO EARTHQUAKE CHARACTERISTICS
(REF. 2-21)

(a) GENERATION AND TRANSMISSION OF SEISMIC WAVES
(REF. 2-6)

(b) LIST OF FACTORS AFFECTING SEISMIC WAVE TRANSMISSION
(MODIFIED FROM REF. 2-6)

FIGURE 2-5. FACTORS AFFECTING GROUND SHAKING AT A SITE

2.2.1 SOURCE MECHANISM

Earthquake-induced ground motions are initiated when sudden slip takes place along an active fault and releases stresses that have been stored in the earth's crust. Recent advances in knowledge have indicated that this slip process can have an important effect on ground motions in the general vicinity of the causative fault (Ref. 2-22).

When an earthquake occurs, its effect can depend on the type of faulting that generates the seismic waves. Three general types of faults -- strike-slip, reverse, and normal -- are illustrated in Figure 2-6. Pure examples of these fault types rarely occur; rather, the displacement of the fault has components both parallel and normal to the fault trace. In addition, the fault slip typically occurs along an irregular surface rather than a plane, and there may also be branch and secondary faulting (Ref. 2-23).

Some of the other source parameters that affect the nature of the seismic waves generated by fault slippage are shown in Figure 2-5b. Those of particular engineering importance have been described in Reference 2-17 and are summarized as follows:

Stress Drop -- When fault slippage occurs, the stress in the rock across the fault changes from an initial value, σ_1 , to a lower stress, σ_2 . The magnitude of this stress drop, $(\sigma_1 - \sigma_2)$, has an effect on the amplitude of the seismic waves generated.

Total Fault Displacement -- The magnitude and energy released by the earthquake probably are related to the total displacement along the fault, larger fault displacements being related to large magnitudes and to large amounts of energy released. A larger displacement also results in a longer duration of fault movement and in larger amplitudes of long period waves.

LEFT-LATERAL MOVEMENT

RIGHT-LATERAL MOVEMENT

(a) STRIKE-SLIP FAULT (1906 CALIFORNIA EARTHQUAKE)

HIGH-ANGLE REVERSE FAULT
(1971 SAN FERNANDO EARTHQUAKE)

LOW-ANGLE REVERSE FAULT
(1964 ALASKA EARTHQUAKE)

(b) REVERSE FAULTS

(c) NORMAL FAULT (1954 DIXIE VALLEY-FAIRVIEW
PEAK, NEVADA, EARTHQUAKE)

FIGURE 2-6. FAULT TYPES

Length of Fault Break -- An earthquake of large magnitude is generated by fault rupture of long length. The length of the fault break also can have a significant effect on the duration of strong shaking, which in turn can be correlated with the damage potential of the ground motions. Differences in the slipped-fault length of the 1964 Alaska earthquake (Magnitude 8.4 and slipped-fault length of 450 mi) and the 1971 San Fernando earthquake (Magnitude 6.5 and slipped-fault length of 12 mi) contributed to significant differences in the nature of the ground shaking generated by these two events.

Single or Multiple Fault Breaks -- Recent studies of the 1964 Alaska earthquake and the 1940 Imperial Valley earthquake have indicated that these events were generated by a series of multiple fault breaks, rather than a single break (Ref. 2-6). It is apparent that the nature of ground shaking will be different if the seismic waves are generated by multiple fault breaks and erratic propagation of the slip boundary, rather than by a uniform propagation of the slipped area with relatively smooth slippage of all points along the fault plane.

Fault Shape -- A large-magnitude earthquake generally occurs on an elongated fault area whose length greatly exceeds its width. Small earthquakes, below Magnitude 6, usually occur over a slipped area whose dimensions are about the same in all directions -- although some smaller earthquakes, such as the 1966 Parkfield event, have had narrow, elongated fault areas. The dimensions of the slipped area will have a strong influence on the nature of the ground shaking, particularly with regard to frequency content, duration, and felt area of the earthquake.

Proximity of Fault Plane -- In the central region of an earthquake, the proximity of the fault rupture to the ground surface has a strong influence on the ground shaking. If the upper portion of rupture is well below the ground surface, the shaking will not be the same as it would be if the rupture reached the ground surface. Especially in the near-field, the ground acceleration amplitudes depend more on the proximity to the faulting process than on the magnitude.

2.2.2 TRANSMISSION PATH FROM SOURCE TO SITE

When slippage occurs along a fault plane, energy is released in the form of seismic wave emissions as well as heat. These seismic waves propagate away from the causative fault through the geologic layers. The primary types of waves that transmit energy from the causative fault are P-waves (dilatational components) and S-waves (shear components). Secondary waves may exist at interfaces between different strata (Stoneley waves) and at the ground surface (Rayleigh waves). Horizontally propagating Rayleigh waves are characterized by long period motions, and are usually important only at distances greater than those of interest for strong motion earthquakes (Ref. 2-3).

The seismic waves that propagate from the source to the site are influenced by numerous parameters related to the geometry and material properties of the transmission path, as indicated in Figure 2-5. Along this path, both P-waves and S-waves are reflected and refracted as they encounter the different layered interfaces, resulting in interference with or strengthening of the waves as they propagate from the source of energy release. Irregularities in the transmission path, such as variations in surface topography and discontinuities in the subsurface layer geometries and properties, greatly complicate the reflection and refraction processes affecting these waves.

In addition to undergoing modifications as various materials are encountered along the transmission path, amplitudes of the seismic waves are also modified as a result of geometric spreading effects. Additional

attenuation occurs because of the dissipative properties of the subsurface soil and rock materials. Because of this latter effect and the general nonlinear characteristics of these subsurface materials, the seismic wave energy is also dependent on the amplitude and frequency content of the wave motions generated at the source.

2.2.3 LOCAL SITE CONDITIONS

The seismic wave energy arriving at a site is carried primarily in the P-waves and S-waves transmitted through subsurface rock materials. In order for these waves to reach the ground surface, they must be transmitted through the overlying soils. These soil deposits can consist of complex sequences of sand, gravel, clay, and silt, all of which are highly nonlinear and inelastic. As a result, local soil conditions can exert an important influence on the seismic waves transmitted from the rock to the ground surface. This potential influence of local soil materials has been recognized and successfully evaluated at a number of sites (Refs. 2-24 and 2-25, for example).

It has been shown that for a site with horizontal, well-defined layers and with incoming seismic waves in bedrock having wavelengths comparable to the horizontal extent of the site, the primary mechanism for transmitting horizontal ground motions to the ground surface is vertically-incident shear waves (Ref. 2-25). Furthermore, these shear waves are not strongly influenced by the effects of inhomogeneities in the subsoils or by slight irregularities in the geometry of the layer boundaries. For this case, one-dimensional analytical techniques can be used to estimate horizontal earthquake motions over the entire depth of the soil profile. If these conditions are not met, however, the effects of the overlying soil on the ground motions are considerably more complex and the resulting soil response analyses

must be treated as a two-dimensional wave propagation problem. A more detailed discussion of soil profile modeling is contained in Chapter 4.*

Some of the variables that influence the effects that local soil conditions will have on the ground shaking at a site are outlined in Figure 2-5b. Among the more important variables are the following:

Depth to Rock -- Correlations between structural damage and the depth to rock at sites with similar soil conditions have indicated that high-rise buildings with low natural frequencies have sustained the greatest damage when sited on deep soil deposits. In contrast to this, shorter and stiffer buildings with higher natural frequencies have been most vulnerable when located on shallower deposits. This suggests that the depth to rock affects the frequency content of the seismic waves transmitted to the ground surface. (See Sec. 2.4.2 for an example correlation of structure damage with depth to rock.)

Nonlinear Stress/Strain Properties of Soil Materials -- Past studies of soil materials have indicated that their effective modulus decreases and material damping increases with increasing soil strain levels (Ref. 2-25). Because of these strain-dependent material properties, the amplitudes and frequency content of surface ground motions have been correlated with the strength of the subsurface input motions; i.e., an increase in the amplitudes of the input motions causes a reduced characteristic frequency of the ground surface motions and a reduced amplification (or even an attenuation) of the input motions as they are transmitted through the overlying soil layers (Ref. 2-5).

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*It is noted that site-dependent vertical ground motions are automatically computed when two-dimensional analysis techniques are used. One-dimensional site models for predicting vertical motions on the basis of P-wave propagation have occasionally been used; however, these procedures are still under development and involve considerable uncertainty, primarily in the definition of appropriate soil properties.

Geometry of Layers and Ground Surface -- As discussed above, the degree of inclination of the layers or the presence of significant topographic features can greatly influence reflection and refraction processes and the complexity of waves transmitted to the ground surface. For example, if the soil layers are sharply inclined, horizontal ground motions can no longer be attributed solely to vertically propagating S-waves but instead arise from complex interactions of P-waves, S-waves, or secondary waves.

2.3 EARTHQUAKE-MOTION DATA BASE

Data presently available to serve as a basis for estimating earthquake induced ground motions at a nuclear plant site consist of observational and instrumental records of past earthquakes, artificial earthquakes, empirical scaling relationships based on past records, and various soils and geologic investigations. Each of these data sources is described below, to provide background for subsequent discussions of the factors involved in the estimation of seismic input criteria (Sec. 2.5).

2.3.1 RECORDS OF PAST EARTHQUAKES

Strong motion accelerographs of the general type currently in use were first designed and installed in 1933, and the Long Beach, California, earthquake of that year provided the first strong-motion accelerograms (Ref. 2-26). Prior to that time, earthquake records consisted solely of documented effects of damage and personal accounts of felt effects, which, in turn, could be converted to intensities and peak accelerations. There are numerous uncertainties associated with conversions of this type, some of which are discussed in Section 2.1.3.

Since 1933, accelerograph networks around the world have expanded to include almost 3000 operating instruments, most of which are in Japan, the United States, Russia, and Yugoslavia. The United States has about 700 accelerographs, most of which are located in California because of the predominant seismic activity in that region, as shown in Figure 2-7 (Refs. 2-27, 2-28). These instrument locations represent a diverse set of ground and geologic conditions; some are on rock and many others are on soil sites corresponding to a wide variety of local geologic, topographic, and subsoil conditions. In important seismic regions, arrays of accelerographs have been installed to study local effects. For example, there are several horizontal arrays of accelerographs extending across the San Andreas Fault in California, as well as downhole vertical arrays to measure earthquake motions at various depths (Ref. 2-6). Unfortunately, as shown in Figure 2-7, there is relatively little instrumentation in the central and eastern United States, where the majority of present or prospective nuclear plant sites are located.

In the United States, approximately 250 records of strong motion shaking from some 20 to 30 earthquakes have been obtained on the ground or in the basements or lower floors of buildings.* Because of the predominant seismic activity in the far western states, California in particular, most of the records of ground shaking in the United States are from that area. For example, about 100 such records were measured during the 1971 San Fernando earthquake. Smaller numbers of strong ground-motion records were obtained from earthquakes in the Borrego Mountains (1968), Lytle Creek (1970), San Francisco (1957), Kern County (1952), Parkfield (1966) and various earthquakes near El Centro.

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*The total number of strong motion records obtained to date (1975) in the United States is approximately 400, with about 250 records measured during the San Fernando earthquake and about 110 additional records obtained from the Borrego Mountains earthquake (Refs. 2-6, 2-26). However, many of these records were obtained in the mid-to-upper floors of buildings and therefore include the response characteristics of these structures. Such records cannot be considered as part of a usable data base that represents *ground* shaking.

(a) DISTRIBUTION OF DAMAGING EARTHQUAKES (AS OF 1970)

(b) LOCATIONS OF STRONG MOTION ACCELEROGRAPHS (AS OF 1972)

(c) LOCATIONS OF NUCLEAR PLANT SITES (AS OF MARCH 1974)

FIGURE 2-7. COMPARISON OF ACCELEROGRAPH LOCATION WITH EARTHQUAKE DISTRIBUTION AND NUCLEAR PLANT SITE LOCATIONS (REFS. 2-27, 2-28)

The above set of measured records provides a vital source of information regarding the basic characteristics of earthquake ground motions. However, there are a number of important gaps in this present-day collection of strong motion accelerograms. First, the shaking in the immediate vicinity of the causative fault in a great earthquake (Magnitude 8 or more) has never been recorded. Also, relatively few recordings in the epicentral areas of Magnitude 7 earthquakes have been made in the United States (Olympia, Washington [1949] and Kern County, California [1952]). The same is true of potentially damaging shocks of lesser magnitude; only a few lower-magnitude earthquakes, such as San Fernando (1971) and Imperial Valley (1940), have resulted in strong motion records very near the causative fault.

Another important gap in the present collection of strong motion records is associated with the complete lack of such measurements in the eastern and central United States. Because most of the recorded seismic data for the United States have come from California, the characteristics of the accelerograms will tend to reflect the particular seismologic and geologic conditions of that region. These general conditions in California are substantially different from those of the east and midwest. For example, as shown in Figure 2-8, felt areas from eastern earthquakes are more extensive than for comparable sized earthquakes occurring in California (Refs. 2-29, 2-30) and, in addition, the frequency of occurrence of large earthquakes is much greater in California. Furthermore, the current tectonic forces in California are dominated by the San Andreas fault system, a strike-slip fault more than 600 mi long that represents a zone of slippage of the Pacific plate northward past the North American plate. No comparable fault system exists elsewhere in the United States and, in fact, there is a general lack of understanding of the earthquake producing mechanisms in regions other

(a) INTENSITY MAPS (REF. 2-29)

(b) INTENSITY ATTENUATION CURVES (REF. 2-30)

FIGURE 2-8. FELT AREAS OF VARIOUS REGIONS OF UNITED STATES AS INDICATED BY INTENSITY MAPS AND ATTENUATION CURVES

than California, owing to a scarcity of records and to a general lack of surface faulting when earthquakes have occurred.* Further research and measured data is required to evaluate the extent to which earthquake motions in the East and Midwest might differ from those in California because of differences in the geology and tectonics of these areas. Until such additional information is provided, the current collection of ground motion records, primarily from California, must serve as the best available basis for defining strong ground-shaking characteristics throughout the country.

2.3.2 ARTIFICIAL EARTHQUAKE RECORDS

As discussed in Section 2.3.1, there is a lack of strong motion measurements close to epicentral areas of potentially damaging earthquakes of moderate-to-large magnitude. Furthermore, variations in the characteristics of ground motion records from comparable earthquakes around the world have been noted. As a result, a much larger sample of records is required to adequately define the properties of strong ground shaking.

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*Some notable recent studies of the seismic characteristics of regions other than California have been made, although these studies do not yet confirm the causes and mechanisms of earthquakes in these regions. For example, Sbar and Sykes have suggested that a number of the seismic belts in the East may be due to intraplate tectonic effects that are associated with the same forces that cause movement of the North American plate and sea floor spreading beneath the Atlantic Ocean (Ref. 2-31). Nuttli (Ref. 2-32) and Mitchell (Ref. 2-33) indicate that the slower rate of attenuation of earthquakes in the East and Midwest could be related to the dispersion of short-period Rayleigh waves.

To partially fill the gaps in the current collection of strong motion records, researchers have utilized the theory of stochastic processes to generate ensembles of simulated earthquakes. These artificial earthquake records embody the basic properties of strong-motion records, as indicated from available data and from engineering judgment regarding subsequent events. Both stationary and nonstationary random processes have been used to generate these artificial records, although nonstationary random processes are generally recognized as being required particularly for modeling smaller earthquakes (Refs. 2-34, 2-35, 2-36).

Perhaps the most widely used artificial earthquake records are those developed by Jennings, et al., at the California Institute of Technology (Ref. 2-34). The formulation of these records is based on the observation that, at distances sufficiently far from the causative fault so that the details of the fault displacement are not reflected in the ground motion measurements, a component of recorded ground acceleration usually appears to be a random function having the following characteristics:

- a. The time-dependent variation of earthquake ground motions consists of an initial segment of rapid increase to high amplitudes, a segment of uniformly strong shaking at these high amplitudes for a certain duration, and an attenuating final portion of the motion.
- b. The frequency content of the motion described by a smoothed Fourier spectrum indicates the frequency ranges in which the major part of the energy occurs.
- c. As illustrated by digital computation, the structure modal responses excited by the ground accelerations also consist of a time during which the amplitude of vibration initially rises, followed by a duration of strong vibrations and a period of gradually attenuating vibration.

The procedure used by Jennings, et al., to derive artificial earthquake records is to first generate a random white noise process whose duration is judged consistent with the magnitude and fault distance for the earthquake type being simulated. This random process is filtered to produce the desired frequency content as specified by comparisons with existing real records. The filtered function is then shaped by multiplying by an amplitude-shaping function that produces an initial rise in the accelerogram, an attenuating tail, and a strong central portion. Artificial earthquake records developed by this process are described in Figure 2-9.

This method of generating earthquake motions enables the engineer to construct artificial records that represent earthquakes of specified magnitudes at specified distances from the fault. In addition, he can generate several ground motions that correspond to specified input criteria and can thereby perform response calculations that account for statistical variations (Ref. 2-17). However, most artificial records generated to date do not consider local site conditions and are generally based on past records that, as discussed in Section 2.3.1, reflect ground shaking primarily in California and the Far West.

2.3.3 EMPIRICAL DATA FOR ESTIMATES OF GROUND SHAKING CHARACTERISTICS

The determination of seismic input criteria for a nuclear plant site involves, at several stages, the use of empirical data to guide the specification of the criteria ground motion characteristics. For example, curves derived from these data serve as an aid in specifying: (1) the peak accelerations and duration of the subsurface rock motions (for input into the soil response analyses of Step 3, Fig. 1-1); (2) the peak ground accelerations (for use in scaling the site-matched records and the Regulatory Guide 1.60 spectra in Steps 5 and 6, respectively); and (3) the duration of strong shaking at the ground surface.

Simulated Earthquake Type	Type of Earthquake Motion	Strength of Earthquake	Total Duration, sec	Duration of Strong Motion, sec
A	Represents upper bound for ground motions expected near causative fault during earthquake having Richter Magnitude of 8 or greater.	150 percent as strong as average spectrum intensity of 1940 El Centro records.	120	29
B	Models shaking close to fault in Magnitude 6-7+ earthquake (e.g., 1940 El Centro and 1952 Taft).	Equal to average spectrum intensity of 1940 El Centro records.	50	11
C	Simulates motion expected in epicentral region of Magnitude 5-6 shock (e.g., 1957 San Francisco and 1935 Helena, Montana).	Equal to average spectrum intensity of 1957 Golden Gate records.	12	2
D	Models shaking close to fault of very shallow earthquake of low-to-moderate magnitude (e.g., 1966 Parkfield, and 1972 Stone Canyon).	Maximum acceleration scaled to be equal to that of Parkfield record (0.5 g).	10	0.5

(a) DESCRIPTION OF ARTIFICIAL RECORDS

(b) ARTIFICIAL EARTHQUAKE TIME HISTORIES

FIGURE 2-9. ARTIFICIAL EARTHQUAKE RECORDS DEVELOPED BY JENNINGS, ET AL. (REF. 2-34)

It should be noted that these empirical curves are based on a limited number of available earthquake records that, as discussed in Section 2.3.1, correspond primarily to events in California and some other far western states. Also, the curves generally involve correlations with magnitude and causative fault distance,* although the interrelationship between these parameters and ground motion characteristics cannot yet be defined with certainty. Therefore, empirical curves for estimating ground motion characteristics must be used with care and interpreted with sound engineering judgment.

In the paragraphs that follow, several of the more significant empirical relationships between earthquake ground motion characteristics and the magnitude, and causative fault distance are summarized. The ground motion characteristics included in this summary are limited to the peak acceleration, velocity, and displacement and the duration of strong shaking; empirically obtained spectrum shapes for use in describing the frequency content of earthquake motions, also an important characteristic, are discussed in detail in Chapter 4.

2.3.3.1 Peak Ground Surface Motions

Empirical studies of strong motion data on the ground surface have produced a number of ground-motion attenuation curves. These curves typically consider peak horizontal acceleration with only limited studies of peak velocity or displacement. Few data on vertical motions are available, and current practice in nuclear plant siting typically assumes the peak vertical acceleration to be some fraction, usually $2/3$, of the peak horizontal ground acceleration.

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*There are also several correlations between peak ground acceleration and intensity, and between intensity and distance for various parts of the United States. These correlations have been described in Sections 2.1.3 and 2.3.1.

Perhaps the most widely used curves for attenuation of peak ground motions have been those developed by Housner in 1965 (Ref. 2-18) and shown in Figure 2-10a. These curves provide interrelationships between peak horizontal acceleration, Richter magnitude, and causative fault distance. Some more recent studies of peak horizontal ground accelerations by Cloud (Ref. 2-37) and Page, et al. (Ref. 2-20) are shown in Figures 2-10b and 2-10c. In these curves, it is noted that Cloud's results do not indicate a strong dependence of peak acceleration on magnitude, whereas the data of Page, et al. and Housner suggest a reasonably clear increase in acceleration with increasing magnitude.

There have been relatively few attenuation curves for peak horizontal velocity or displacement. Page, et al. have recently attempted to correlate these quantities with Richter magnitude and causative fault distance but, as shown in Figure 2-11, their results show wide scatter.

2.3.3.2 Peak Accelerations at Rock Outcrops

Since subsurface bedrock motions typically serve as input to soil response analyses, they must be estimated from the geologic and seismologic studies of the region. In the absence of subsurface bedrock motion measurements, rock outcrop records are often used as guides for defining the peak accelerations and general characteristics of these input motions. Empirical data based on observed acceleration-attenuation characteristics of rock outcrop motions are typically applied for this purpose.

The work of Schnabel and Seed probably represents the most widely utilized data for defining the acceleration level of rock motions at a site (Ref. 2-38). Their work is based on the characteristics of observed and computed rock motions in the western United States and Mexico, and is summarized in the form of curves that relate peak acceleration to the earthquake magnitude and causative fault distance. Curves of this type were first developed by Seed and his associates in Reference 2-39; these curves were updated by Schnabel and Seed to include measurements from the 1971 San Fernando earthquake and other recent events, as well as calculations of

(a) HOUSNER RESULTS (REF. 2-18)

(b) CLOUD RESULTS (REF. 2-37)

(c) PAGE, ET AL., RESULTS (REF. 2-20)

FIGURE 2-10. EMPIRICAL SCALING RELATIONSHIPS FOR PEAK HORIZONTAL GROUND ACCELERATION

(a) VELOCITY

(b) DISPLACEMENT

FIGURE 2-11. CORRELATION OF PEAK HORIZONTAL VELOCITY AND DISPLACEMENT WITH FAULT DISTANCE AND RICHTER MAGNITUDE (REF. 2-20)

rock outcrop motions. Results of the Schnabel-Seed study are provided in the form of average values of peak acceleration-attenuation rates (Fig. 2-12a). In addition, curves indicating the scatter in these attenuation rates are also shown; this scatter reflects the influence of different source mechanisms, geologic environments, and travel paths (Fig. 2-12b).

Rock-motion attenuation data for the central United States have been estimated by Nuttli (Refs. 2-29, 2-40). In the absence of ground motion measurements in this region, these data are based on highly approximate correlations between particle velocity and intensity for hard rock materials. Results for different frequency components of the seismic waves are shown in Figure 2-13. Unfortunately, these data are not in a form that can be readily used for scaling input to conventional soil response analyses; however, they are included in this summary, since they represent one of the few known attempts for estimating earthquake motion characteristics in regions of the United States outside of California and the Far West.

2.3.3.3 Scaling Relationships for Duration of Strong Shaking

As discussed in Subsection 2.1.3, the duration of strong shaking is an important measure of the potential damage to structures caused by earthquake motions. Most studies to date have recognized that the duration can be related to the earthquake magnitude, since the magnitude is a measure of the stress drop and of the length and displacement of the slipped region of the fault which, in turn, strongly affects the strong motion duration. Housner, in 1965, (Ref. 2-18) investigated duration vs. magnitude correlations for sites centered near the causative fault. Studies by Bolt (Ref. 2-19) and Kobayashi (Ref. 2-41) have developed similar duration vs. magnitude correlations incorporating more recent data. These results are summarized in Figure 2-14.

In addition to investigating magnitude correlations, Bolt studied the duration of strong shaking for various frequency components of the seismic wave (Ref. 2-19). By filtering horizontal motion records from nine earthquakes, he found that the greatest duration of strong shaking for frequency components was in the 1-to-5 Hz range (Fig. 2-15a), although Kobayashi (Ref. 2-41)

(a) AVERAGE VALUES

(b) RANGES OF VALUES

FIGURE 2-12. ATTENUATION OF PEAK HORIZONTAL ROCK ACCELERATIONS IN WESTERN UNITED STATES (REF. 2-38)

(a) BOUNDARIES OF SEISMIC REGIONS

(b) 0.3 HZ SIGNALS

(c) 1 HZ SIGNAL

(d) 3 HZ SIGNALS

FIGURE 2-13. FREQUENCY DEPENDENT ROCK ACCELERATIONS FOR CENTRAL UNITED STATES (REF. 2-40)

(a) HOUSNER RESULTS
(REF. 2-18)

(b) BOLT RESULTS (REF. 2-19)

(c) KOBAYASHI RESULTS
(REF. 2-41)

FIGURE 2-14. DURATION OF STRONG SHAKING VERSUS EARTHQUAKE MAGNITUDE

(a) EFFECT OF FREQUENCY COMPONENT (REF. 2-19)

(b) EFFECT OF DISTANCE TO SLIPPED FAULT OR EPICENTRAL DISTANCE FOR WESTERN UNITED STATES EARTHQUAKES (REF. 2-20)

FIGURE 2-15. CORRELATIONS OF DURATION OF STRONG SHAKING WITH FREQUENCY AND DISTANCE TO FAULT OR EPICENTER

suggests these results may be affected by the properties of the filter used by Bolt.

Some studies of the effect of causative fault distance on the duration of strong shaking have been made, but no definite trends can be observed. Page, et al., in a study of western and Alaskan earthquake data, suggest that for a given magnitude, strong motion duration decreases with increasing distance (Ref. 2-20). However, as shown in Figure 2-15b, the data are widely scattered in the Magnitude 6.0 to 6.9 range and are relatively sparse for higher magnitudes. In contrast to this, Nuttli suggests that central United States earthquakes do not have the same duration characteristics as western earthquakes; in fact, because of increased dispersion effects and weak attenuation of surface wave energy, Nuttli suggests that the duration of strong shaking may actually be increased with increasing distance for central United States earthquakes (Ref. 2-40). The disparity of results of Page, et al., and Nuttli suggests that the duration of strong shaking may be more dependent on regional characteristics than on causative fault distances. Further research is required in this area.

2.3.4 SOILS AND GEOLOGIC INVESTIGATIONS

When predicting earthquake motions based on local soil properties at a site, it is important to consider soil and geologic characteristics at locations of prior strong motion measurements and the degree to which these characteristics have been studied in the past. The paragraphs that follow briefly describe the availability of soils and geologic data from prior earthquake recording sites.

As indicated in Section 2.3.1, earthquake records obtained to date represent motions from a diverse set of ground and geologic conditions. This is probably because many engineers are primarily interested in estimating earthquake ground motions at sites of important structures, rather than in developing data on a particular geologic environment that might provide an improved basis for calculating ground motions at a number of sites. Only recently has significant earthquake engineering research been applied to

detailed studies of the influence of local site conditions on earthquake ground motions. As a result, there is now a scarcity of reliable soils and geologic data at sites of strong motion recording instruments.

Of the approximately 400 records obtained in the United States, nearly half have been recorded in upper levels of high-rise buildings; therefore, they include the effects of the building response as well as the response of the local soil deposits. Most of the remaining strong motion records have been measured on soil deposits and therefore include the modifying influence of local soils. Only a few measurements have been obtained on rock outcrops. No subsurface measurements of ground shaking during strong earthquakes have been obtained.

For most accelerograph stations located on soil deposits, data on soil properties and geologic conditions are either nonexistent or limited in nature. Many stations have no available data, and the site description simply consists of the statement "Soil of undetermined character." Where data describing geologic and soil conditions at accelerograph stations are available, they have been obtained from a variety of studies, most of which were conducted without concern for the behavior of the site under seismic loads. For example, the primary soils data sources thus far have been boring logs and foundation reports associated with mineral exploration and deep water wells, or civil works projects and large buildings located near or at the accelerograph station. However, because of the nature of these soils data, and the specific needs they are intended to fulfill, they are typically neither complete nor accurate enough to adequately describe the dynamic behavior of the soil profile.

Because of the above shortcomings in the knowledge of site conditions at accelerograph stations, a number of recommendations have been made to various state and national legislative bodies regarding appropriate

remedial measures. For example, the Joint Committee on Seismic Safety in the State of California has recommended to the California State Legislature that

...greater effort and appropriate instrumentation should be devoted to studies of the effects of topography and the character of geologic material on the distribution and amplitude of strong ground motion. (Ref. 2-42)

Unfortunately, most recommendations along these lines have either not been implemented or are being carried out on a low priority basis. One of the few known cases in which subsurface instrumentation has been used to evaluate dynamic effects of soil materials was at Union Bay in Seattle, Washington. At this site, a three-level accelerometer station was installed in 1965 and was subsequently used to measure soil transmission characteristics and motions from minor earthquakes and distant underground nuclear events (Ref. 2-43).

In addition to studies of the type conducted at Union Bay, some attempts have been made to describe subsurface soil conditions at various recording stations. For example, site descriptions for a number of seismoscope and accelerograph stations were published in 1962 on the basis of shallow borings, well drilling logs, and other less detailed sources of subsurface data (Ref. 2-44). Although a general indication of site condition is provided, much of the data is unreliable for determining dynamic site response characteristics. A similar catalog of strong motion site conditions was prepared following the 1971 San Fernando earthquake (Ref. 2-45). Shallow refraction surveys were conducted at many sites and provided substantially improved small strain shear wave velocity data for depths up to 100 ft. However, shear wave velocities and other soil properties obtained for deeper layers at these sites are less reliable and are still not adequate for dynamic response analyses. A more recent and comprehensive program for conducting soils investigations at accelerograph stations is presently being carried out by a Shannon & Wilson-Agbabian Associates joint venture for the USNRC. This consists of field investigations and laboratory tests of the subsurface soil materials at selected stations. In addition, background information

regarding the regional geology and seismic history for these various sites is also included. Only now are initial results of this investigation becoming available (Ref. 2-46).

2.4 CASE HISTORIES OF PAST EARTHQUAKES

In this section, descriptions of the observed damage and measured records of past earthquakes are provided to illustrate events whose effects were either strongly or weakly dependent on local soil conditions. The 1906 California, 1967 Caracas (Venezuela), and 1940 Imperial Valley earthquakes have been selected to represent the numerous examples of each type that can be cited.

2.4.1 1906 CALIFORNIA EARTHQUAKE

The earthquake of 18 April 1906 is one of the greatest to have occurred in California. The epicenter of this earthquake was at the south end of Tomales Bay along the San Andreas fault, and visible slippage along the fault extended from Shelter Cove in Humboldt County to San Juan Bautista in San Benito County, an overall distance of more than 270 mi. Horizontal displacement offsets along the fault were substantial, with the west side moving north as much as 21 ft at Point Reyes and 14 ft at Point Arena. Vertical upthrows were also quite large, measuring as much as 6 to 8 ft along the east side of the fault at Shelter Cove. Damage caused by this great earthquake has been estimated to be as much as one billion dollars, and an estimated 700 people were killed. A large part of this loss was due to major fires within San Francisco, which resulted from broken gas lines and the lack of water caused by ruptured water pipes (Ref. 2-3).

The city of San Francisco was the most heavily populated area affected by the 1906 earthquake. Because of its large population, its variety of geologic and soil conditions, and its close proximity to the San Andreas fault, the damage induced by the earthquake within the city has been of special interest. The paragraphs that follow deal specifically with the damage caused in this area.

The soil conditions within San Francisco consist primarily of Pleistocene and recent deposits of dune sand, water-laid sand, mud, and clay as well as extensive deposits of slopewash, artificial fill, alluvium, and landslide debris. Depths to rock within the city are quite variable. Outcrops of sandstone, shale, and serpentine rock of the Franciscan Formation exist throughout the city, especially along a shear zone extending from Hunters Point to Fort Point (Ref. 2-47).

A comprehensive study of the 1906 earthquake (Wood, Ref. 2-48) has correlated the damage induced in San Francisco with local geologic and soil conditions. These correlations are clearly shown in the maps in Figure 2-16. The maps indicate that, except for the on-land portion adjacent to the San Andreas fault (southwest corner of maps), the most severe damage occurred in the artificial fill materials along San Francisco Bay. Within the city, however, the observed damage varied greatly over short distances. The areas of minimum damage were associated with rock outcrops, while adjacent areas of strong-to-very strong shaking were associated with sand and alluvial deposits. As a result, it is seen that for this particular earthquake and the site conditions within San Francisco, local geology had an important effect on the damage that resulted in the area.

2.4.2 1967 CARACAS, VENEZUELA, EARTHQUAKE

On 29 July 1967, Caracas, Venezuela, and nearby areas were shaken by an earthquake with a Richter Magnitude of 6.3, centered about 60 km northwest of downtown Caracas. There were no strong motion measurements obtained from this earthquake; however, various estimates have placed the duration of strong shaking at about 20 sec and a peak acceleration of about 0.08g in Caracas Valley (Ref. 2-49).

(a) SIMPLIFIED GEOLOGIC MAP OF THE SAN FRANCISCO PENINSULA DURING TIME OF 1906 EARTHQUAKE

(b) INTENSITY MAP OF SAN FRANCISCO PENINSULA--1906 EARTHQUAKE

FIGURE 2-16. RELATION BETWEEN GEOLOGIC CONDITIONS AND INTENSITY IN SAN FRANCISCO DURING 1906 EARTHQUAKE (REF. 2-48)

The local soil conditions in the Caracas Valley consist of alluvial fans, deposited by streams draining the steep southern slopes of the Sierra de Avila mountains and intermixed with alluvial stream deposits laid down by the Rio Guaira. This has resulted in generally good foundation conditions, consisting of interbedded sands, gravels, and stiff clays. Dynamic characteristics of these soils are fairly uniform within the Caracas Valley, although depths to rock vary considerably in this area (Refs. 2-49, 2-50, 2-51).

The effects of this earthquake were felt in all parts of the Caracas Valley; however, the damage and loss of life were especially severe in the Los Palos Grandes area and in Caraballeda, along the sea coast. Since the quality of construction was about the same throughout the area, other possible causes for this localization of damage were examined. Because prior investigations of other areas of the world had indicated strong correlations between building damage and local site conditions, it was suspected that a similar correlation might account for the structural damage trends observed during the Caracas earthquake. The local site condition of primary importance was considered to be the depth to rock, since the dynamic properties of the soil materials themselves were reasonably uniform throughout the area.

To investigate this effect, R. V. Whitman and H. B. Seed were commissioned to make detailed studies of the area. The results of their studies were reported to the Venezuelan government (Refs. 2-50, 2-51) and were summarized in a joint paper (Ref. 2-52). Their studies attempted to correlate structural damage in Caracas and Caraballeda with local site effects through statistical correlations of observed damage and analytical investigations of the effects of local site conditions on earthquake motions in the area.

The results of the Seed-Whitman evaluations of observed damage are summarized in Figure 2-17. These results clearly show a distinct general pattern of damage, with a great increase in damage vulnerability of multi-story structures constructed on soil deposits of substantial depth.

To further analyze the correlation of apparent damage with local geology in Caracas, a series of analytical studies of site-dependent ground motions was carried out. For those sections of the city underlain by horizontal soil layers and base rock boundaries, analyses were carried out using a one-dimensional, vertically oriented, lumped-mass model of a soil column. Numerous soil column depths were modeled in order to represent specific sites within the Caracas area. For those sections of the city with sloping boundaries, two-dimensional finite element models of the soil profiles were utilized. Various time histories scaled to an appropriate acceleration level represented the subsurface bedrock motions.

The results of these analytical studies indicated peak ground surface accelerations of about 0.06g to 0.08g, which corresponded closely to the original estimates provided in Reference 2-49. Of even more importance, however, was the close correlation between fundamental periods of damaged structures, as estimated from shaker tests of similar undamaged buildings, and computed fundamental periods of the soil deposits at the locations of the damaged structure (Fig. 2-17c). This correlation has indicated the undesirability of constructing buildings that have fundamental periods similar to the soil deposits on which they are located, unless special precautions are taken in the structural design.

In summary, statistical and analytical assessments of building damage caused by the 1967 Caracas earthquake showed important interrelationships between site conditions and structural damage in Caracas. In addition, the site-dependent calculations carried out by Seed and Whitman have provided important insights into these interrelationships and have indicated a basis for determining improved design requirements that might serve to reduce similar earthquake-induced structural damage in the future.

(a) MAP OF CARACAS SHOWING DEPTH OF SOIL AND LOCATION OF DAMAGED BUILDINGS

STRUCTURAL DAMAGE INTENSITY VS. DEPTH TO ROCK

STRUCTURAL DAMAGE INTENSITY VS. FUNDAMENTAL PERIOD OF SOIL DEPOSITS

(b) EFFECT OF SITE CONDITIONS ON STRUCTURAL DAMAGE

FIGURE 2-17. RELATION BETWEEN LOCAL SOIL EFFECTS AND STRUCTURAL DAMAGE--1967 CARACAS, VENEZUELA EARTHQUAKE (REFS. 2-49, 2-52)

2.4.3 1940 IMPERIAL VALLEY EARTHQUAKE

On 18 May 1940, a damaging earthquake occurred in the Imperial Valley of southeastern California near the town of El Centro. The earthquake had a local magnitude of 6.3 and was centered along the previously unknown Imperial fault. Accelerograms recorded in El Centro during this earthquake are among the strongest ground motions yet recorded; therefore they are of special engineering significance.

The Imperial Valley is bounded by the Chocolate and Cargo Muchacho Mountains to the northeast and by the Peninsular ranges to the southwest. The valley is a basin about 6 km deep that is occupied by the sedimentary delta of the Colorado River. This delta consists of interbedded sands, gravels, silts, and clays that were derived from rocks of the Colorado plateau eroded during the cutting of the Grand Canyon and the uplift of the Kaibab Plateau; these rocks were deposited in the basin during the past four million years (Ref. 2-53).

Trifunac and Brune, in their study of the 1940 Imperial Valley earthquake, have indicated that the event is characterized by a number of multiple fault breaks, with at least four events recorded at El Centro during the first 25 sec (Ref. 2-53). The epicentral locations of these events are shown in Figure 2-18a, and their times of occurrence are clearly evident in the measured accelerograph records (Fig. 2-18b).

Response spectra for the two horizontal components of motion recorded at El Centro are shown in Figure 2-18c (Ref. 2-54). These spectra contain important amplitudes of motion at various periods that cannot be correlated with existing information regarding the dynamic properties of the underlying soil materials.

The above evidence suggests that the motions recorded at El Centro were strongly influenced by the travel path, the source mechanism, and the complex seismic waves generated along the causative fault. Local soil conditions may possibly have affected some aspects of these records; however, other factors must clearly be considered in determining the overall character of the ground motions in this case.

2.5 SUMMARY AND DISCUSSION OF SEISMIC INPUT CRITERIA DETERMINATION

To provide a proper perspective regarding the problems involved with defining seismic input criteria, the background information contained in this chapter is summarized and discussed in terms of the recommended procedure depicted in Figure 1-1 of Chapter 1.

An important point from this chapter is that the ground shaking at a site is an extremely complex process that cannot be predicted with certainty from current technology. The many factors that affect ground shaking, such as the source mechanism, transmission path, and local site conditions, combine in different ways for different earthquakes. To illustrate this point, measurements and damage observations from past earthquakes have indicated a strong dependence on local soil conditions in some cases and on the characteristics of the source mechanism and transmission path in other cases.

A second important point from this chapter is that the data base of prior strong motion measurements is limited. Nearly all of the relatively few measurements obtained in the United States have been from earthquakes in California and the Far West and, in fact, a large proportion of these records is from a single event, the 1971 San Fernando earthquake. Despite this limited data base, available strong motion records are an invaluable source of information and guidance for the prediction of future earthquake ground motions at a site; however, at the same time, the practicing engineer should

be aware of their limitations so that ground motion prediction methods based solely on these past records, such as the approach in the USNRC Regulatory Guide 1.60 (Ref. 2-55), are not misapplied or their results misinterpreted.

To supplement the existing library of strong-motion records and to estimate ground motions representative of the actual characteristics of the site, soil response analysis procedures can be applied. With properly utilized models and carefully interpreted results, these analytical techniques can serve as important aids in the ground motion prediction process. However, as discussed subsequently in Chapter 4, the results of these analyses must be used with judgment because of the inevitable difficulties in determining the subsurface input motion characteristics and in establishing the properties of the soil profile at substantial depths. These difficulties are due in large part to the general lack of subsurface soil response records and the lack of adequate soil profile information at recorder locations. Additional data along these lines are required to provide more definitive evaluations of site-dependent effects on actual strong motion accelerogram measurements.

The factors described above suggest that a rational approach for the determination of seismic input criteria at a nuclear plant site should not be based solely on either strong motion records or soil response analyses by themselves. Instead, a flexible approach should be adopted that utilizes measured records and analyses to their best advantage for the particular site in question. To account for possible effects of source mechanism, transmission path, and local soil amplification characteristics, the use of parametric soil response analyses together with site-matched records and the results of statistical studies of past strong motion measurements (i.e., Regulatory Guide 1.60) should be considered at some stage in the criteria determination process. The final selection of the seismic input criteria must then be based on sound engineering judgment regarding the applicability of each of these possible sources of data at the particular site under investigation.

The approach shown in Figure 1-1 for determination of seismic input criteria has been established with the above considerations in mind. The remainder of this report contains detailed discussions, evaluations, and an example problem that will aid the engineer as he proceeds through each step of the approach. When interpreted in light of the information contained in this chapter and throughout this report, the approach should provide a sound basis for selection of seismic input criteria at a nuclear plant site.

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CHAPTER 3

GEOTECHNICAL INVESTIGATIONS

The principal geotechnical investigations and procedures required to obtain the licensing of a nuclear plant are outlined in the USNRC criteria document denoted as Appendix A, 10 CFR, Part 100, and entitled "Seismic and Geologic Siting Criteria for Nuclear Power Plants," (Ref. 3-1). These procedures and criteria are based on experience accumulated by the USNRC, the utility industry, and their consultants in evaluating the seismic and geologic conditions at a large number of sites throughout the United States (Refs. 3-2, 3-3).

This chapter summarizes the geologic, seismologic, and site soils investigations and testing aspects required for use in deriving site-dependent vibratory ground motion criteria in accordance with Appendix A, 10 CFR, Part 100. To provide this information, the chapter is divided into six sections. The first section describes the general procedures for performing the geotechnical investigations. The next section contains background information, such as basic definitions and the preliminary planning required for the geotechnical investigations. Descriptions of the investigations of regional geology and seismology and the resulting definition of the most significant earthquake events are provided in the next three sections. The final section describes site geology and soils investigations, including field investigations, laboratory tests, and the development of the generalized subsurface profile for the site.

3.1 GENERAL PROCEDURE

To comply with the USNRC regulatory criteria (Appendix A, 10 CFR, Part 100) requires the coordinated efforts of several different disciplines, including geology, seismology, and engineering. Guidance for organizing these efforts is provided in Figure 3-1. In this figure, the geotechnical

FIGURE 3-1. GEOTECHNICAL INVESTIGATIONS FOR EVALUATING GROUND-MOTION CHARACTERISTICS

investigations are divided into three phases: planning the investigations (planning phase), implementing the field and laboratory studies of the region and the site (investigations phase), and using results from these studies to define the seismic input criteria for the site (engineering phase). The planning and investigations phases of Figure 3-1 correspond to Steps 1 and 2 of Figure 1-1, while the engineering phase refers to Steps 3 to 6 of that figure.

3.1.1 PLANNING PHASE

The planning phase comprises the preliminary work needed to develop and coordinate subsequent field and office studies. This includes literature reviews, site inspections, and scheduling of the subsequent work. The detailed planning of the geotechnical investigations is further discussed in Section 3.2.2.

3.1.2 INVESTIGATIONS PHASE

The bulk of the detailed regional and site investigations, laboratory testing, and preliminary estimates of vibratory ground motions at the site are carried out in the investigations phase. These include field and office studies of the regional geology and seismology, and investigation of the local geology and soil engineering properties at the site. The work in each of these categories is closely related and may be carried out concurrently or in sequence, starting with the regional studies.

The regional geologic and seismologic studies involve investigation and evaluation of the physiography, stratigraphy, tectonics, and the geologic and seismic history of the area. These studies identify the seismic events and associated geologic features that together define the most significant earthquakes to be considered in evaluating the site response.

The site geology and soils investigations establish the local geologic conditions, including the soil profile and the rock and soil properties necessary for static and dynamic design requirements. The soil properties determined for static considerations include the unit weight, shear strength and, where applicable, the relative density. The soil properties obtained for dynamic requirements include the shear modulus, damping ratio, Poisson's ratio, and again, the unit weight, shear strength, and mass density. Of these, the shear modulus and damping ratio are particularly important in seismic response calculations. All of these soil properties can be determined from a properly planned field exploration and laboratory testing program which usually consists of borings, geophysical surveys and other field measurements, sampling of the subsurface soil materials, and appropriate laboratory tests.

The determination of vibratory ground motion criteria utilizes the geotechnical investigations as a basis for performing soil response analyses and for selecting suitable site-matched strong motion records. Procedures for evaluation of potential site failure conditions are also required at a nuclear plant site but, as indicated previously, these are beyond the scope of this report.

3.1.3 ENGINEERING PHASE

The engineering phase involves evaluations and engineering studies necessary to determine the ground motion criteria for the site. These particular studies are the primary emphasis of this report and are described in Chapters 4 and 5. This phase also requires engineering studies that are not specifically directed toward vibratory ground motion characteristics, and therefore are not included within the scope of this report. As indicated in Chapter 1, these studies include evaluations of the hydrology of the area; the potential for surface faulting at the site and its possible effects; soil liquefaction; slope instabilities; earthquake-induced floods, tsunamis, and so forth. It is often necessary during this last phase to expand on some geologic aspects or to provide further laboratory data on the engineering properties of the foundation materials at the site.

3.2 BACKGROUND INFORMATION

This section contains background information pertinent to the use of Appendix A, 10 CFR, Part 100 as a basis for performing geotechnical investigations at nuclear plants. This includes (1) basic definitions contained in the USNRC criteria, and (2) the initial planning procedures required for the geotechnical investigations.

3.2.1 DEFINITIONS

The USNRC seismic and geologic siting criteria define two levels of earthquake ground motions for which a nuclear plant must be designed: the Safe Shutdown Earthquake (SSE) and the Operating Basis Earthquake (OBE). These earthquakes are typically obtained by correlating past earthquakes with known geologic structures or -- in the absence of such features -- with tectonic provinces. Definitions of SSE, OBE, capable faults, and tectonic provinces are provided in the paragraphs that follow.

3.2.1.1 Safe Shutdown Earthquake

The Safe Shutdown Earthquake (SSE) represents the maximum possible level of earthquake ground shaking at the site. The hypothetical event probably has not occurred at the site during historic times; instead, it is based on a detailed investigation of the regional and local geology, regional seismology, and characteristics of the underlying soil materials. For this earthquake, the facility should be designed so that all systems necessary to protect the health and safety of the public will remain functional both during and after the event. Although structures and their internal components may suffer severe damage due to the SSE, the design must allow for a safe and orderly shutdown following the seismic event. A lower bound to the peak horizontal ground acceleration induced at the site by the SSE is 0.1g.

3.2.1.2 Operating Basis Earthquake

The Operating Basis Earthquake (OBE) represents the maximum level of ground shaking that can be expected to occur at the site during the operating life of the nuclear plant. This earthquake probably has occurred in the vicinity of the site during historic times and, like the SSE, is based on a detailed investigation of the regional and local geology, regional seismology, and characteristics of the underlying soil materials. For this earthquake, the facility should be designed so that those features of the plant necessary for continued operation without undue risk will remain functional. If vibratory ground motions exceeding those of the OBE occur at the site, the plant must shut down until it is demonstrated to the USNRC that these critical features of the plant have not been damaged. According to current regulations, the maximum horizontal ground acceleration of the OBE must be at least half that of the SSE, with a lower bound peak acceleration level of 0.05g.

3.2.1.3 Capable Faults

The investigation of faults and the faulting potential in the site area are of particular importance in the regional and site geologic investigation. As defined in Reference 3-1, a "fault" is a tectonic structure along which differential slippage of the adjacent earth materials has occurred parallel to the fracture plane. "Surface faulting" is differential ground displacement at or near the surface, caused directly by fault movement. The USNRC criteria indicate that a fault can be considered "capable" of permanent surface displacement during the useful lifetime of a power plant if it is characterized by one or more of the following features:

- a. Movement at or near the ground surface at least once during the past 35,000 years or more than once in the past 500,000 years
- b. Instrumentally determined macroseismicity directly related to the fault

- c. A relationship to an active fault such that movement on the one fault could be reasonably expected to be accompanied by movement on the other

3.2.1.4 , Tectonic Province

A tectonic province is defined as a region characterized by a relative consistency of the geologic structural features contained therein. The boundaries of the province are determined through a regional analysis of the geologic and tectonic environments, *and the seismic history of the area* (Ref. 3-2). As noted subsequently, the concept of a tectonic province has particular importance where earthquake epicenters cannot be related to known faults or to other major specific tectonic structures.

3.2.2 PLANNING OF GEOTECHNICAL INVESTIGATIONS

Prior to undertaking the geotechnical investigations described in Section 3.1, it is necessary to devise a general overall view of the project; outline the technical and managerial tasks; and plan the detailed scope of work. The tasks typically carried out during this phase include the following:

- a. A detailed review of pertinent geologic and seismologic literature regarding the region, the area, and the site. During this phase of the work, appropriate federal, state, and local-university geologists, seismologists and other experts in related fields who might possess unpublished information, or personal knowledge pertinent to the studies, should be contacted. For example, information may be obtained from the following sources:

U.S. Geological Survey

U.S. Corps of Engineers

U.S. Bureau of Reclamation

U.S. Department of Agriculture

U.S. Coast and Geodetic Survey (previous publications)

U.S. Nuclear Regulatory Commission

U.S. Department of Defense

U.S. Department of Housing and Urban Development

National Oceanic and Atmospheric Administration

Earth Resources Technology Satellite data from the
National Aeronautics and Space Administration

State, county, and city agencies

Geology, geophysics, and soils departments of local
universities

Local, private consulting firms

Private industry (oil, gas, water, well drillers)

- b. An inspection of the site and vicinity to gain familiarity with geologic and physiographic characteristics of the site and vicinity
- c. Preparation of a detailed scope of work, time requirements, and cost estimate for each subsequent phase of the project

- d. Preparation of a quality assurance program structured to guide all activities comprising the work, and to inform the owner and the Nuclear Regulatory Commission about controls and inspections governing the quality of work

3.3 REGIONAL GEOLOGY

The geologic characteristics of the region around the site should be investigated in sufficient scope and detail to provide a clear understanding of the physical processes that formed the area, as well as its potential for future seismic activity. The size of the region to be investigated is dependent on the nature of the area surrounding the site. It may be generally limited to a distance of 200 mi around the site, or may include an entire tectonic province.

The geologic investigations normally begin with an evaluation of all available literature in the area. This information is correlated with existing published and unpublished geologic maps to determine the preliminary stratigraphic relationships, tectonic framework, and geologic history. From this study, a preliminary geologic map is compiled. Aerial photographs of the area are then studied and interpreted. Finally, standard geologic field mapping techniques are used with subsurface studies to compile the final geologic map and to establish the physiography, stratigraphy, and structural geologic conditions of the area. These investigations are discussed further in the paragraphs that follow.

3.3.1 PHYSIOGRAPHY

Concurrent with the review of published data, a detailed site and areal reconnaissance of the region should be performed in order to identify important geologic surface features (i.e., land forms, drainage patterns, areas of surface water collection, etc.), and to evaluate the relationship between such features and probable subsurface conditions. The study also should include a consideration of the relationship between the regional and

the site physiography. Topographic maps and aerial photographs are essential to these studies. An end result of this work should be the compilation of a regional physiographic map in which the site and all of the study data are properly identified.

3.3.2 STRATIGRAPHY

The stratigraphic investigations of the region should include a thorough analysis and description of the lithology, composition, mode of occurrence and age of the various geologic units underlying and surrounding the site. Geology maps should be prepared for the region. A stratigraphic column and geologic cross sections that relate regional and local geologic conditions to the site location should also be provided. This information is developed from the study of available literature and, in addition, from field mapping of outcrops, well logs, and special borings made for this purpose.

3.3.3 TECTONICS

Investigations of the tectonics of the region should include identification and evaluation of all tectonic structures within the region surrounding the site, whether buried or expressed at the ground surface. All faults, folds, basins, and domes should be included in this evaluation. Furthermore, in view of the fact that past experience has indicated that the loading effects of dams and reservoirs, the pumping or extraction of subsurface water, or the extraction of minerals can produce earthquakes (Refs. 3-4, 3-5), the possible effects of these man-related activities in the vicinity of the site should be considered.

Of particular importance in this phase of the geotechnical investigation is the identification of all capable faults within 200 mi of the site. The presence of faults in an area may be indicated in various ways. Sometimes they can be observed directly from exposures in the field, or they may be detected during the drilling of wells or borings. Usually, however, their

identification will be based on more indirect evidence. In some cases the presence of faulting can only be inferred from the various surface and sub-surface techniques commonly available. In certain instances it may be impossible to decide whether a fault is actually present. For these cases, other techniques -- such as angled borings, trenching, or tunneling -- must then be utilized to establish that faulting has occurred at that locality. Faults can frequently be detected or inferred from such geophysical investigations as gravity or aeromagnetic surveys, and acoustical or seismic profiles; or by remote sensory techniques such as black-and-white, color, or infrared aerial photography, side-looking airborne radar imagery, and so forth.

The criteria for recognition of faults include: (1) presence of structural or stratigraphical discontinuities; (2) repetition, omission, or offset of strata; (3) abrupt changes in sedimentary facies; (4) visible displacement of veins, dikes, strata, formations, etc.; (5) presence of features characteristic of fault movement, such as drag, slickensides, striations; (6) presence of breccia zones; (7) physiographic evidence; and (8) anomalous ground water conditions. Recent fault traces generally can be recognized in the field or on aerial photographs by linear topographic discordances; contrasts in vegetation that may reflect varying depths to ground water or soil differences across a fault trace; anomalous drainage patterns; or offset man-made structures. The most common fault-break features indicating recent movement along a fault trace are scarps, ridges, notches, trenches, offset streams, lines of springs, and sag ponds and other undrained depressions.

As discussed in Section 3.2, the definition of a capable fault is dependent on whether movement has occurred along the fault in geologically recent times. Therefore, to evaluate these potential fault movements,

particular in a relatively young geologic area, various dating techniques such as those outlined below are typically employed:

- a. The limiting age of 35,000 years commonly can be established either indirectly by geologic methods or absolutely by radio-carbon determinations from organic or other materials that contain radioactive carbon (C^{14}).
- b. Although several geochronologic methods are available to establish a limiting age of 500,000 years, absolute dating frequently is not feasible. Commonly, combinations of the various dating techniques must be used to arrive at a limiting age. The various dating techniques include:

Geologic methods, which include paleontologic and stratigraphic relations, weathering profiles (paleosols), and geomorphic features

Paleomagnetic methods, which are a measure of the remnant magnetism of certain rocks and minerals

Radiometric methods, which measure the radioactive decay of various nuclides that occur naturally in many rocks and minerals (chiefly, potassium argon)

Geochemical methods, which measure the concentration or amounts of specific trace elements or of the oxides of some of the elements (particularly useful when working with basalts and other volcanic rocks)

Once capable faults are identified, it remains to evaluate their potential for producing a major earthquake. For this reason the following

features of those faults with lengths greater than those indicated in Table 3-1 should be determined:

- a. Fault length
- b. Magnitude and nature of displacement
- c. Geologic history of displacements, particularly the age of latest movement
- d. The relationship of the fault to regional tectonic structures

From the results of the regional tectonic investigation, a tectonic map should be prepared that shows the distribution of major faults, folds, warps, and other structural features, defining them in terms of dimensions, attitude, and age. The map should also show areas of actual or potential surface or subsurface subsidence, uplift, or collapse resulting from tectonic or human activities.

3.3.4 GEOLOGIC HISTORY

The final step in the regional geology investigations is to reconstruct the geologic history of the area, with special emphasis on the late Tertiary and younger tectonic history. The geologic history of the region is of prime importance, in that it provides the regional framework essential for establishing the pattern and probable age of major deformations occurring in the region, including the possibility of recent faulting in the area. Reconstruction of the geologic history is based on evaluations of the physiography, stratigraphy, structural geology, and tectonics of the region.

TABLE 3-1. MINIMUM FAULT LENGTH VERSUS
DISTANCE FROM SITE (REF. 3-1)

Minimum Distance from Site, mi	Minimum Fault Length to Be Considered, mi
0 ~ 20	1
20 ~ 50	5
50 ~ 100	10
100 ~ 150	20
150 ~ 200	40

3.4 SEISMOLOGY

The potential for earthquake activity in the vicinity of a proposed nuclear power plant site is determined from investigations of the seismic history and characteristics of the site and the surrounding region, and their relationship to the geologic conditions. These investigations involve a thorough research, review, and evaluation of all historically reported earthquakes that have affected, or that could reasonably be expected to have affected, the site. Listings of each of these earthquakes should be prepared, and should include the following measured or estimated information:

- a. Date of Occurrence. The date of occurrence for each past event provides a basis for estimating earthquake-occurrence rates. In addition, the date of each event can be correlated with population densities, construction practices, and the number of earthquake recording instruments that existed at the time of the event. In this manner, the reliability of the earthquake data can be evaluated.
- b. Epicentral Intensity and Magnitude. The epicentral intensity indicates the effects of an earthquake at its epicenter, whereas the magnitude of a given earthquake provides an indication of the energy released by the event. Prior to 1933 there was a lack of earthquake recording instruments; therefore, most information regarding the size of these earlier earthquake events has been provided in terms of intensities, although equivalent magnitudes have been estimated for major events. For events occurring after 1933, both measured magnitudes and intensities are usually available. In the absence of strong motion measurements, the reliability of either of these parameters as a measure of peak ground motions is questionable (see Chapter 2).

- c. Epicenter Location. The epicentral location of each event, either instrumentally determined or inferred from the observed damage and felt area patterns, should be defined.

Once the above information is obtained for each seismic event, the following data should be compiled:

- d. Epicentral Maps. Two maps should be prepared, each showing the tectonic structural features and earthquake epicenters in the vicinity of the site. One map should show the epicenters of *all* reported earthquakes centered within 50 mi of the site, and the other should provide the epicentral locations of all earthquakes equal to or larger than intensity V (MM) that have been reported within 200 mi of the site.
- e. Correlation of Epicenter Locations and Tectonic Structures. The epicenters, or locations of highest intensity, of historically reported earthquakes should be correlated wherever possible with tectonic structures, any part of which is located within 200 mi of the site. Epicenters, or locations of highest intensity, that cannot be reasonably correlated with tectonic structures should be identified with tectonic provinces, any part of which is located within 200 mi of the site.
- f. Rates of Attenuation with Distance. When the region surrounding the epicenter of a given earthquake is reasonably well populated, isoseismal maps can be compiled, even for some of the early events. From a sufficient quantity of such isoseismal data, it is possible to construct intensity attenuation relationships for the region such as those shown in Figure 2-8 (Chapter 2).

- g. Recurrence Intervals. Where sufficient data are available, the frequency of occurrence of earthquakes of a given size can be determined. These recurrence intervals provide useful supporting data for determining the design earthquake. As described by Richter (Ref. 3-6) recurrence intervals are typically represented by:

$$\log N = a - b I_o$$

where

N = Number of earthquakes per year of epicentral intensity I_o or greater

a, b = Empirical constants that are dependent on the seismicity of the region

Recurrence curves of this type for the entire United States and for California are shown in Figure 3-2 (Ref. 3-7).

- h. Identification of Appropriate Empirical Data. Earthquake events are most typically reported in terms of such parameters as magnitude, intensity, and epicentral distance. As a result, empirical charts relating these parameters to peak acceleration, duration of strong shaking, etc., are available in the literature and must often be used to estimate ground-motion characteristics at nuclear plant sites. Some of these charts are reproduced in Figure 2-2 and in Figures 2-10 through 2-15 (Chapter 2).
- i. Other Data. When available, other relevant data should be reported, such as the duration of strong shaking at various reported locations, the area within which the earthquake was felt, and important damage effects (such as landslides, liquefaction, or subsidence).

(a) United States data, 1534-1965

(b) California data, 1769-1965

FIGURE 3-2. EARTHQUAKE RECURRENCE CURVES (REF. 3-7)

3.5 DEFINITION OF MOST SIGNIFICANT EARTHQUAKES

The purpose of the geologic and seismologic studies of the region and the site is to help establish the Safe Shutdown and Operating Basis Earthquakes according to USNRC criteria requirements (Ref. 3-1). These studies provide the information needed to identify the seismic events and relevant geologic features that together define the most significant earthquakes to be considered in evaluating vibratory ground motion effects at the site.

The most significant earthquakes are determined from an evaluation of the seismology, the geology, and the seismic and geologic histories of the site and surrounding region. The most severe earthquakes associated with tectonic structures or tectonic provinces in the region surrounding the site should be identified by considering the historically reported earthquakes and other applicable relevant factors such as duration and felt areas, damage effects, recurrence intervals, earthquake magnitude vs. fault length, and displacement on the fault (Ref. 3-8). If capable faults exist in the region, the most severe earthquake associated with these faults should be determined by also considering their geologic history. The general procedures for defining the potentially most significant earthquakes are as follows:

- a. Where the historic earthquakes of greatest intensity can be correlated with a fault or tectonic structure near the site, the epicenters of these events are assumed to be situated at the point on the fault or tectonic structure that is closest to the site.

- b. Where the epicenter of a given earthquake of large intensity cannot be reasonably related to a particular fault or tectonic structure, it must be identified with a tectonic province. If the site is also within that province, the earthquake is assumed to occur at the site. Otherwise, the earthquake is assumed to occur at the point along the boundary of the tectonic province that is closest to the site.

Once potential significant earthquake events are located according to either of the procedures indicated above, it remains to determine the basic design parameters required for the seismic response studies. These parameters include a first estimate of the peak ground acceleration, the duration of strong shaking, and peak rock acceleration. They are obtained from both the intensity-attenuation data for the region and from the empirical data that relate these parameters to the magnitude, intensity, and epicentral distance of the earthquake event (see Sec. 3.4). It is noted that final estimates of the peak ground acceleration for the site must be based on the site response analyses (Steps 3 and 4 of Fig. 1-1), as well as on the first estimates of peak acceleration obtained from empirical data.

3.6 SITE GEOLOGY AND SOIL ENGINEERING PROPERTIES

The investigations to be carried out at proposed nuclear plant sites have been described in detail in Reference 3-9. Those studies necessary for evaluating the ground motion characteristics at the site, as summarized in this section, consist of both field and laboratory investigations. Their overall purpose is to define all aspects of the site geology, including the soil and rock profiles and the material properties required for foundation design and for seismic input criteria development. In general, these procedures include determination of the soil and rock classifications, the thickness and lateral distribution of the soil strata, groundwater levels, and the engineering properties of the bedrock and of each soil layer down to bedrock or to relatively firm materials.

3.6.1 FIELD DATA

Field investigations at the site of nuclear plants are carried out for the following purposes:

- a. To obtain information and data regarding the classification, lateral distribution, stratification, geologic structure, and engineering properties of the soil deposits and bedrock underlying the site; and to relate these data to the geology as determined by local and regional investigations
- b. To obtain soil samples and representative rock cores for identification, classification, and laboratory testing
- c. To establish the elevation and variation of the groundwater table at the site as required for foundation studies
- d. To assist in evaluating the dynamic elastic properties of the overburden soils and the velocity of wave propagation in the underlying rock

These investigations are carried out using borings, trenches, and geophysical techniques as required by the characteristics of the particular site. Each of the exploratory procedures, together with the principal data derived from them, is briefly described in the paragraphs that follow. For purposes of this report, the site investigations are essentially restricted to those associated with the soils at the site. For detailed descriptions of the procedures and resulting information, the reader should consult Reference 3-9.

3.6.1.1 Borings

In the investigation of a potential nuclear power plant site, the drilling of borings and recovery of samples are generally the most practical and direct methods of investigating the subsurface conditions. The number and depth of borings to be made at a site, under normal static load

considerations, depends upon the characteristics of the plant structures and the competence and complexity of the soil at each specific building location. Structural characteristics that must be considered when planning the borings include the loads, size, type of construction, the intended use and performance criteria for the structures, and the elevations of the various foundation elements. Since a nuclear power plant usually consists of a number of structures of various sizes and loads, borings are normally required at each specific structure location and throughout the site area; these borings should define the subsurface conditions within the area of foundation influence and should verify the continuity of these conditions across the site. At the smaller and less important facilities, one or two borings may suffice, whereas many more will be required for the larger, more important structures; the actual number depends, of course, on the variation and complexity of the subsurface conditions. Usually, one or more borings should extend into bedrock or into dense "rock-like" soils at depths of several hundred feet in order to establish soil and bedrock conditions for response calculations. These deep borings generally identify and verify the competence and establish the continuity of the bedrock and deeper soil deposits, which are typically not penetrated by the shallower borings made in the usual static load foundation investigation.

To obtain information for ground response analyses, it is necessary to determine certain local subsurface soil parameters, which can best be evaluated by means of the borings noted above. In addition to field identification and classification of the soils, the parameters to be determined by borings include: (1) the distribution and thicknesses of the soil strata at the site, (2) the *in situ* relative density of granular deposits, (3) the groundwater conditions, and (4) the bedrock depth. A brief discussion of these parameters follows.

- a. Thickness of Soil Strata. To define layer thickness within a deposit, samples are generally taken at intervals of 5 ft in the upper 100 ft of depth. Smaller intervals are frequently necessary above 50 ft, while larger intervals are satisfactory at greater depths. Complex sites with thin lenses of materials may require sampling at more frequent intervals.
- b. Relative Density. Various types of penetration-resistance tests are used as an indirect method to determine *in situ* relative density of granular soil deposits. In the United States, the most widely used method is the Standard Penetration Test (SPT). Data from this test may be related to the relative density of the deposit by means of the Gibbs and Holtz correlation. This procedure is primarily useful for indicating average trends and ranges of values and is less meaningful for precisely indicating a specific value for any given sample. For a more complete treatment of this subject, refer to Reference 3-9, Chapters 3 and 8.
- c. Groundwater Conditions. For evaluating the groundwater conditions, it is generally satisfactory to install water measuring devices in approximately half of the exploratory borings depending, of course, on the actual site conditions; this is further described in Reference 3-9, Appendix A. Groundwater elevation data enter indirectly into site response calculations because they influence the effective confining pressure which, in turn, generally affects both the damping and stiffness (modulus) properties of the soil.
- d. Bedrock Depth. The depth to bedrock or to rock-like materials at a particular site is required for use in response calculations. This parameter is determined quite readily from borings to depths of at least 300-to-500 ft. Below these depths, geophysical refraction surveys have been found to be an

acceptable procedure to determine the depth to rock. Results of such surveys, however, should be verified with boring and geological data. For soil response calculations, the approximate accuracies of bedrock depths shown in Table 3-2 are desirable. Additional information regarding current drilling and sampling methods, inhole test procedures, and available groundwater measuring devices, may be found in Reference 3-9, Appendix A.

3.6.1.2 Trenches

Test trenches or pits are used to provide a continuous or limited exposure of the ground along a given line or section. Depending upon the extent of the investigation required and the site conditions, trenches can be excavated either by pick and shovel methods, or by bulldozers, ditching machines, backhoes, or draglines. Trenches permit visual inspection and recording of soil and rock strata and are used in the geologic investigation to determine the presence or absence of faulting and, in addition, to aid in locating and identifying faults and faulted zones, where present. They also aid in providing large, undisturbed samples of overburden materials and permit the determination of in-place densities at shallow depths. The location, number, and depth of trenches in nuclear power plant site explorations vary largely according to prevailing geologic conditions; as a result, no general guidelines for planning trenching operations can be formulated. In actual practice, however, the depth of trenching is limited to about 15 to 50 ft, depending upon the site conditions.

3.6.1.3 Geophysical Explorations

Geophysical explorations usually consist of seismic refraction velocity measurements and are used to determine the depth to bedrock, to delineate the general distribution and competence of the overburden strata, and to estimate dynamic elastic properties of the overburden soils and underlying rock. These measurements generally supplement field and laboratory

TABLE 3-2. ACCURACY DESIRED FOR BEDROCK DEPTH DETERMINATION

Approximate Bedrock Depth Zone, ft	Approximate Accuracy ft
0 - 100	5
100 - 200	5 - 10
200 - 500	20 - 50
500 - 1000	100
> 1000	±200

information provided by borings and trenches. Three classes of geophysical explorations -- the seismic refraction survey, inhole methods, and the electrical resistivity method -- are briefly described in the paragraphs that follow. A more complete treatment of these procedures can be found in Reference 3-9.

Seismic Refraction Survey

The seismic refraction survey consists of placing geophones along the ground surface at a given interval (usually 25, 50, or 100 ft) in a line, and at several selected locations along the same line. Then explosive charges are fired and the arrival times of the resulting wave patterns are recorded simultaneously at each geophone with the aid of an oscilloscope. The total length of the survey line and the size of charge govern the depth of penetration of the seismic waves, while the spacing of the geophones controls the detail with which the overburden strata can be delineated. A short line may be only 500 ft, while a long line may be 2500 ft.

With the longer lines of geophones indicated above, bedrock can be detected with reasonable accuracy down to depths approaching 1000 ft. Results of such surveys, however, should be verified with (1) reported nearby water- or oil-well drill logs, if available; (2) geological records, maps, or profiles throughout the area; and (3) borings at the site, when necessary. The depth to bedrock can be determined with an accuracy that, typically, decreases with increasing depth, as indicated in Table 3-2.

Inhole Surveys

Other more specialized seismic survey methods are used to obtain specific soil properties for dynamic response analysis. These methods represent inhole procedures (conducted inside a boring), and consist of the uphole, downhole, and crosshole techniques. The uphole and downhole surveys measure the seismic velocities of soil up or down a borehole. For the uphole survey, successive charges are exploded in the hole at selected

intervals (5 to 10 ft) or at soil layer changes, and the wave velocities are measured by means of geophones at the surface. In the downhole survey, a string of geophones is placed in the borehole (again at selected intervals) and either (1) a charge is exploded adjacent to the hole; or preferably (2) a sledgehammer blow is struck horizontally on a vertically-oriented, rigid steel plate, embedded at shallow depths. For either method of applying energy, the downhole velocities are measured for each depth interval selected. In the crosshole technique, successive charges are exploded at selected depth intervals in a central borehole; and seismic velocities are recorded laterally by strings or geophones in adjacent boreholes, spaced at various distances around the central boring. These techniques are discussed more fully in Reference 3-9, Section 4.2.1.

Another technique used for inhole surveys utilizes an acoustic logging system, such as the 3-D Velocity Log. This technique provides for a continuous downhole or uphole log of pressure waves, from which compressional and shear wave velocities and elastic properties can be computed.

Of the three basic geophysical velocity test methods, the downhole and the crosshole techniques are the two methods most widely used for the determination of the *in situ* shear modulus. These tests are not limited to any particular type of soil and, theoretically, can be conducted at almost any depth to which the boring can be advanced. Under practical conditions, however, the shear modulus measurements are rarely reliable beyond depths of 200 to 300 ft. At these depths, borehole surveys to determine the correct lateral position of the borings are particularly important in the interpretation of results from crosshole methods.

Electrical Resistivity Method

The electrical resistivity method is the only other geophysical method of significance to civil engineering projects. In this, electrodes are inserted in the ground surface in close proximity, an electric current

is induced, and the resistance between the electrodes is measured. The resistivity method is useful for defining the boundaries between soft clays and organic deposits of low resistivity, and materials of higher resistivity, such as sand, gravel, or bedrock. Materials having low resistivities can be detected even if they underlie those of high resistivities. Unfortunately, boundaries between soft clay and an organic soil, stiff clay and soft clay shale, or loose sand and coarse-grained sandstone cannot usually be detected.

3.6.2 LABORATORY DATA

The earthquake-induced response of soil deposits, for cases involving no residual displacements, is governed primarily by the dynamic shear moduli and the damping characteristics. Other pertinent properties include unit weight, Poisson's ratio, bulk modulus and strength under static and dynamic loading. Also, a number of other soil characteristics, such as consolidation and permeability, are routinely obtained in foundation investigations; however, because these properties are not directly utilized in earthquake response analyses, they are not included in the discussion that follows.

The following paragraphs provide guidance in selecting the most appropriate tests to determine the material properties required in site response analyses. A tabular summary of the recommended test methods is presented in Table 3-3. Again, complete descriptions of these tests may be found in Reference 3-9.

a. Unit Weight. The unit weight of the soil materials is used directly in response analysis calculations. Average values must be assigned to each layer within the soil deposit. Because variations in this property of ± 5 pcf produce negligible change in the resulting output, the selection of the testing method is not critical.

TABLE 3-3. DETERMINATION OF SOIL MATERIAL PROPERTIES REQUIRED FOR VIBRATORY GROUND MOTION INVESTIGATIONS

Property	Type of Test	Soil Materials	Test Method Most Commonly Used	Reference 3-9 "State-of-the-Art" Report	
				Section	Page
Classification	Field	All types	Visual inspection	3.2	51
	Laboratory	Granular	Visual inspection; grain-size; natural water content	3.3.1	61
		Cohesive	Visual inspection; Atterberg limits; natural water content		
Unit Weight	Field	All types	In-place density	--	--
	Laboratory	All types	Undisturbed specimen	3.3.2a	66
Relative Density	Field	Granular	Standard penetration	3.2.2	53
	Laboratory	Granular	Undisturbed specimen, if obtainable	3.3.2c	69
Shear Modulus*	Field	All types	Geophysical	4.2.1	78
	Laboratory	All types	Triaxial; simple shear; resonant column	4.2.2	89
Damping*	Laboratory	All types	Triaxial; resonant column	4.3.2	113
Poisson's Ratio	Field	All types	Geophysical	4.4	116-117
	Laboratory	All types	Triaxial; usually estimated	--	--
Bulk Modulus	Laboratory	All types	Triaxial; not commonly measured	4.4.2	117
Static Strength	Laboratory	Cohesive	Unconfined compression	3.3.2b	67

*See Reference 3-9, Chapter 5, for discussion of dynamic properties of soils.

b. Shear Modulus. The shear modulus is strain-dependent and is usually determined by both field and laboratory testing techniques. Merits and limitations of the various approaches are discussed in Reference 3-9, Chapter 4. Geophysical methods employing inhole techniques are currently the most widely utilized field approach for measuring *in situ* shear moduli; in the laboratory, the resonant column and the cyclic triaxial tests are the predominant methods.

c. Damping. Material damping properties for use in response calculations are presently determined using laboratory techniques; no test now exists for determining usable data *in situ*. The resonant column and cyclic triaxial tests are presently most commonly used. Like the shear modulus, this property is also strain dependent.

d. Poisson's Ratio. Although a number of laboratory and field tests can be used to obtain values of Poisson's ratio, either at low strains or at near-failure conditions, it is customary in ground response calculations to use representative values ranging from 0.50 for saturated cohesive materials, to 0.3 to 0.35 for granular soils with low degree of saturation.

e. Bulk Modulus. The bulk modulus is required as input to two-dimensional models for determining the seismic response of soil deposits. In practice, this property is typically not measured; rather, it is usually determined from the shear modulus and from Poisson's ratio. However, where required, the bulk modulus can be measured in the laboratory using the triaxial test and other less commonly applied test methods, as discussed in Reference 3-9.

f. Static Shear Strength. In relation to ground response, the undrained shear strength for clay soils is of particular interest, since it provides a convenient parameter for normalizing the dynamic, strain-dependent shear modulus. Its justification and use for this purpose is described and illustrated in Section 5.4 of Reference 3-9. Typical values for clays may be measured *in situ* with depth by using the vane test or, alternatively, by

testing undisturbed samples with any of the conventional shear-test devices in the laboratory. For cohesionless soils, variations in strength (expressed as the effective angle of internal friction) have been found to have only a minor influence on the dynamic soil properties and thus on the computed ground response. Rather than strength, the *in situ* compactness or relative density, as discussed previously, has a much more significant influence on ground response.

g. Dynamic Shear Strength. Under certain conditions, it may be desirable to determine the dynamic shear strength under cyclic loading to evaluate the liquefaction potential for sand deposits and to evaluate strength reductions for clay deposits. However, because the dynamic shear strength of sands and clays is not directly used in the computation of vibratory ground motions, the measurement of this parameter is not described in this report.

3.6.3 GENERALIZED SITE SOIL PROFILE

The site investigations provide the information necessary to establish a generalized subsurface soil profile for the site. This includes a simplified soil column down to bedrock or "firm ground," which defines each of the soil layers by classification and thickness, as determined from the borings. In this regard, broad descriptions, such as "sand," "gravel," "silt," or "clay," are desirable for indicating the major differences in classification and engineering properties. Each soil layer in the site profile is then assigned the engineering properties determined from site investigations. As previously noted, these properties include the unit weight, the relative density for granular soils, the static shear strength for clays, the shear modulus, and the damping ratio. The shear modulus and damping are strain-dependent properties; therefore, the field and laboratory investigations must be planned to determine these values for the anticipated strain levels. Because of the present limitations in the current technology for defining these strain-dependent properties, data from a number of

investigations have been compiled into curves that relate the various factors influencing these parameters (Ref. 3-9). A close approximation of the appropriate shear moduli may be obtained by determining the moduli from the shear wave velocity profile for the site (at very low strain levels) and by using the published curves to reduce these values for other strain levels. To use the curves, relative densities are required for the granular soils, and static shear strengths are required for the clays. Similar curves are also available for approximating the damping ratios.

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CHAPTER 4

CURRENT TECHNOLOGY FOR ESTABLISHING EARTHQUAKE GROUND MOTION CRITERIA

Basic procedures for establishing earthquake ground motion criteria for nuclear plant sites can be divided into two categories, site independent and site dependent. Site-independent methods consist of the use of standardized spectrum shapes obtained from statistical studies of past earthquake records, typically without regard to differences in characteristics of the sites from which the records were obtained. The current USNRC Regulatory Guide 1.60 is based on an approach of this type (Ref. 4-1). Site-dependent methods utilize site-matched records, which consist of past earthquake motion measurements at sites similar to that being considered for the nuclear plant; and soil response analyses, which use appropriate mathematical models to calculate earthquake-induced ground motions at the site.

As indicated in Section 2.5, the input criteria selection process should utilize site-dependent techniques in conjunction with site-independent methods in order to (1) consider the response of the underlying soil materials and thereby satisfy requirements of Appendix A, 10 CFR, Part 100 (Ref. 4-2); and, at the same time, (2) incorporate the insight gained from studies of past earthquake ground motion records. Therefore, the purpose of this chapter is to describe how these techniques are applied and to assess their relative advantages and limitations. Such information provides important background information for applying Steps 3 through 6 of the recommended procedure for selection of seismic input criteria at nuclear plant sites (Fig. 1-1).

The remainder of this chapter consists of four sections. The first (Sec. 4.1) describes site-independent methods that have been used in the past, with particular emphasis on the spectrum shapes used in Regulatory Guide 1.60. The second and third sections describe site-dependent techniques, including the utilization of site-matched records (Sec. 4.2) and the application of

soil response analyses (Sec. 4.3). The final section (Sec. 4.4) summarizes this technology and the merits and limitations of site-independent and site-dependent methods.

4.1 SITE-INDEPENDENT TECHNIQUES

Site-independent methods for developing seismic input criteria in the form of standardized spectrum shapes were first introduced by Housner in 1959 (Ref. 4-3). Since then, other standardized spectrum shapes have been developed and applied to nuclear plant sites, such as those of Newmark-Hall (Ref. 4-4) and Regulatory Guide 1.60 (as described in a paper by Newmark, Blume, and Kapur, Ref. 4-5). These site-independent techniques are based on studies of numerous strong motion records (Table 4-1), and represent the most widely used approach for developing seismic input criteria.

The above spectrum shapes are classed as site independent, since they are derived from an ensemble of accelerograms that represent a variety of seismologic, geologic, and local soil conditions. These shapes can be adjusted by an experienced investigator to account for particular conditions at a site, but rational adjustment procedures are not available in the current technical literature.

To provide a sound basis for comparing site-independent spectrum shapes to site-dependent results (as required under Step 6 of Fig. 1-1), an understanding of the rationale behind these spectrum shapes is essential. Therefore, brief descriptions of the three most widely used spectrum shapes -- those of Housner, Newmark-Hall, and Regulatory Guide 1.60 -- are provided in this section. For additional information, the reader is encouraged to consult the references cited in the paragraphs that follow.

4.1.1 HOUSNER PROCEDURES

The Housner spectrum shapes were first provided in 1959 as normalized acceleration and velocity spectra plotted to a linear scale (Fig. 4-1). These spectra were obtained as smoothed average response curves developed

TABLE 4-1. BASIS FOR DEVELOPMENT OF SITE-INDEPENDENT SPECTRA

Earthquake Data				Number of Components Considered in Spectrum Determination				Estimated Site Characteristics (Ref. 4-8)	
Location of Epicenter	Date	Magni- tude	Recording Station	Housner (Ref. 4-3) ¹	Newmark/Hall (Ref. 4-4)	USNRC Regulatory Guide 1.60 (Ref. 4-1) ³		Depth to Rock, ft	Site Classi- fication ⁴
						Newmark (Ref. 4-12)	Blume (Ref. 4-13)		
Lower Calif.	1934	6.5	El Centro	2	Correlations provided by Garcia and Roesset (Ref. 4-11) ²		2	100	Stiff
Helena	1935	6.0	Helena				2	Rock	Rock
Imperial Valley	1940	6.3	El Centro	2		3	2	100	Stiff
Wester Wash.	1949	7.1	Olympia	2			2	420	Deep coh.
Northwest Calif.	1951	5.8	Ferndale			3		500	Deep coh.
Kern County	1952	7.7	Taft	2			2	Rock	Rock
			Hollywood Storage Building: Basement Storage Lot			3 3		200 200	Stiff Stiff
Eureka	1954	6.6	Eureka			3	2	500	Deep coh.
			Ferndale			3		500	Deep coh.
El Alamo, Baja Calif.	1956	6.3	El Centro			3		100	Stiff
Daly City	1957	5.3	Golden Gate Park			3	2	Rock	Rock
Hollister	1961	5.6	Hollister			3		--	--
Puget Sound	1965	6.5	Olympia				2	420	Deep coh.
Parkfield	1966	5.6	Tembler				1	Rock	Rock
			Cholame-Shandon #2				2	150	Stiff
			Cholame-Shandon #5				2	100	Stiff
Peru	1966	7.5	Lima				2	--	--
Borrego Mt.	1968	6.5	El Centro			3		100	Stiff
Japan	1968	7.8	Tikachi-Oki				2	Deep	Deep coh.
San Fernando	1971	6.5	Pacoima Dam			3		Rock	Rock
			Orion			3	2	800	Deep coh.
			Castaic			3	2	60	Stiff
			Bank of California			3	2	70	Stiff
			Sheraton Universal				2	--	Stiff

¹Based on mean of spectra from two horizontal components of motion only.

²Garcia and Roesset (Ref. 4-11) showed close correlation between Newmark/Hall spectra and average of envelope spectra from horizontal motions at El Centro (1940), Helena (1935), Taft (1952), Olympia (1949), and Golden Gate (1957).

³Based on mean-plus-one standard deviation statistical level from ensemble of spectra indicated. Separate spectrum shapes developed for horizontal and vertical motions.

⁴Site Classifications in Ref. 4-8: (a) Rock; (b) Stiff, i.e., firm soils less than about 150 ft deep; (c) Deep cohesionless, i.e., sandy soil deposits with depths exceeding 250 ft; (d) Soft-medium, i.e., soft to medium-stiff clays with associated sand and gravel strata (Note: Soft-medium soil sites were investigated in Ref. 4-8, but were not included in development of above site-independent spectra).

(a) VELOCITY SPECTRA (ARITHMETIC PLOT)

(b) ACCELERATION SPECTRA (ARITHMETIC PLOT)

(c) TRIPARTITE SPECTRA SCALED TO ZERO PERIOD ACCELERATION OF 0.1G

FIGURE 4-1. HOUSNER RESPONSE SPECTRA (REF. 4-3)

from the two horizontal components of motion recorded for each of four large earthquakes experienced in the western United States (El Centro 1934, El Centro 1940, Olympia 1949, and Taft 1952) (Refs. 4-3, 4-6, 4-7). Magnitudes assigned to these earthquakes varied from 6.3 to 7.7, and distances from the epicenters to the recording stations ranged from 10 to 35 mi. (Note: The focal depth of the Olympia earthquake, however, was approximately 45 mi.) Details of the soil profiles at the recording stations are not well defined, but it is currently accepted that the four stations represent a variety of site conditions ranging from deep cohesionless soil deposits to stiff soil and rock sites (see Table 4-1 and Ref. 4-8).*

A unique feature of Housner spectra is that the normalized curves are intended to be scaled by a factor based on the spectrum intensity (see Sec. 2.1.4.2) rather than by the peak ground acceleration. For example, the Housner spectra can be scaled to correspond to the average horizontal ground response of any of the four major earthquakes considered; this is carried out by multiplying the ordinates of the Housner spectra by the factors given in Figure 4-1b for the respective earthquakes. Spectrum intensities developed by Housner for zero damped and 20% critically damped spectra for 20 different earthquakes are given in TID 7024 (Ref. 4-6).

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*Much of this discussion of site-independent methods and site-matched records centers on the classification of various strong-motion accelerograph stations as rock, stiff soil, deep cohesionless, or soft-to-medium soil sites, as defined in Reference 4-8 and Table 4-1. These classifications are based on the best available information regarding the subsurface conditions at these sites; however, as noted in Section 2.3.4, data describing the soil and rock materials at these stations are often limited. A program of field explorations and laboratory tests of the soil materials at these stations is required to provide more definitive information along these lines.

Ordinarily, the strength of the input criteria motions for a nuclear plant site is defined in terms of peak ground accelerations. Therefore, it is not always convenient to utilize response spectrum scaling factors that are based on spectrum intensities. As an alternate approach, it is recognized that the response acceleration curves given in Figure 4-1b converge to the maximum horizontal ground acceleration at zero period of vibration; therefore, the curves given in Figures 4-1a and 4-1b can be scaled linearly by the ratio of the peak ground acceleration for the site to the zero-period spectral acceleration given in Figure 4-1b (4 ft/sec^2 or $0.124g$). On this basis, the response spectra given in Figure 4-1c were constructed for $0.1g$ maximum horizontal ground acceleration and have been presented on a tripartite plot. These curves are quite useful for scaling to other maximum horizontal ground acceleration levels and will be used in later comparisons. However, the reader should note that the peak acceleration is not a completely adequate measure of the strength of the ground motions (Sec. 2.1.4), and that scaling linearly by maximum ground acceleration is not a procedure advocated by Housner (Ref. 4-9). This scaling procedure has been adopted here solely for convenience.

In his 1959 paper, Housner also presents the spectra comparisons indicated in Figure 4-2 to portray the effects of distance and earthquake magnitude on the resulting spectra (Ref. 4-3). Housner points out that higher-frequency seismic waves from earthquakes are attenuated more rapidly with distance than are lower-frequency waves. Thus, the high-frequency end of the spectrum is accentuated at close-in recording stations and is depressed at stations located a great distance from the center of energy release. As a result, curve *a* in Figure 4-2 represents the smoothed spectrum for the Taft 1952 earthquake, located 25 mi from the fault; and curve *b* represents the smoothed spectrum derived from the Hollywood Storage Building records for the same earthquake. The site of the latter record is located some

FIGURE 4-2. HOUSNER COMPARISONS OF VELOCITY SPECTRA
(REF. 4-3)

70 mi from the causative fault. In addition, Housner indicates that for ground motions recorded close to the source of a small earthquake, the high-frequency response will be accentuated. Therefore, curve *c*, the smoothed average spectrum for the Golden Gate 1957 record, is provided for comparison with curve *a*. From this, it is seen that variations from a standardized site-independent spectrum may occur due to effects of the size of the earthquake and the distance from the epicenter of the earthquake to the site.

4.1.2 NEWMARK-HALL PROCEDURES

In a paper authored by Newmark in 1967, it was noted that reasonably good estimates of the peak ground acceleration, velocity, and displacement are needed to develop reliable response spectra (Ref. 4-10). From this, and based on studies of response spectra derived from the 1940 El Centro North-South accelerogram (which at that time was the strongest record available from a major earthquake), Newmark and Hall observed that strong motion response spectra could be related to the peak ground acceleration, velocity, and displacement over certain well-defined frequency ranges (Ref. 4-4). This observation, and resulting evaluations of the degree of amplification of ground motion spectra within each frequency range, provided the basis for the formulation of the Newmark-Hall standardized spectrum shapes.

To define an appropriate spectrum shape from estimated values of the peak horizontal acceleration, velocity and displacement for a given site, Newmark and Hall proposed the spectral amplification factors listed in Figure 4-3a. These amplification factors and peak ground motion estimates are used to construct the Newmark-Hall spectra, in the manner indicated in Figure 4-3b. It is noted that, except at the constant displacement portion

Percent of Critical Damping	Amplification Factor for		
	Displacement	Velocity	Acceleration
0	2.5	4.0	6.4
0.5	2.2	3.6	5.8
1	2.0	3.2	5.2
2	1.8	2.8	4.3
5	1.4	1.9	2.6
7	1.2	1.5	1.9
10	1.1	1.3	1.5
20	1.0	1.1	1.2

(a) SPECTRAL AMPLIFICATION FACTORS

(b) CONSTRUCTION OF NEWMARK-HALL FIVE PERCENT DAMPED RESPONSE SPECTRUM

(c) DAMPED RESPONSE SPECTRA

FIGURE 4-3. NEWMARK-HALL RESPONSE SPECTRA AND SPECTRAL AMPLIFICATION FACTORS (REF. 4-4)

of the spectrum, the frequency bounds of the various regions of the spectra are established at multiples of f_1 , the frequency at the intersection of the peak spectral velocity and acceleration. More details on the construction of the Newmark-Hall spectra are provided in Reference 4-4.

As part of their ground motion study, Newmark and Hall suggested that a "standard" earthquake be used whenever specific site information is not available. The standard earthquake has peak ground motions approximately 50% greater than those measured at El Centro (1940), with maximum values of ground acceleration, velocity, and displacement of 0.5g, 24 ips, and 18 in., respectively. On the assumption that these values are proportional to one another regardless of the level of ground shaking, maximum ground velocities and displacements can be computed that are consistent with any specified value of ground acceleration. For example, for an earthquake having a peak ground acceleration of 0.1g, consistent values for maximum ground velocity and displacement scaled from the standard earthquake would be 4.8 ips and 3.6 in., respectively. Horizontal response spectra developed from these peak ground motions according to Newmark-Hall procedures are shown in Figure 4-3c.

It has been specifically noted above that the "standard" earthquake values for maximum ground acceleration, velocity and displacement are, according to Newmark and Hall, for use when specific site information is *not* available. For this case, they did suggest adjustments in their spectral amplification factors based on *general* site conditions, and indicated that the difference in the amplification factors for velocities and accelerations between rock and soft soil sites may be as great as a factor of two. However, no direct procedure for making these adjustments is suggested. Obviously, the investigator must use judgment or must have other sources of information to make these adjustments (Ref. 4-4).

Insight into the interpretation of the Newmark-Hall spectra is provided by a subsequent study by Garcia and Roesset, in which the horizontal response spectra developed by Newmark-Hall methods are compared with spectra derived from accelerograms obtained during the El Centro (1940), Taft (1952), Helena (1935), Golden Gate (1957), and Olympia (1949) earthquakes. The results of this study indicate that the Newmark-Hall spectra correspond reasonably well to average values of peak spectral amplification ratios derived from the above records (Ref. 4-11). Subsequent experience has suggested that the Newmark-Hall spectra correspond to about the 84% probability level from strong motion records.

The studies by Newmark and Hall also provide a basis for estimating criteria spectra for vertical ground motion. These spectra are based on the vertical peak motions being two-thirds of the horizontal peak motions when fault movements are horizontal, and being equal to the horizontal peak motions when the fault movement involves large vertical components. The spectral amplification factors given in Figure 4-3a are then applied to these peak motions to develop the vertical response spectra. The same frequency bounds used to define the various regions within the horizontal response spectra also apply to the vertical spectra.

4.1.3 REGULATORY GUIDE 1.60 PROCEDURES

Two independent studies of the statistical properties of response spectra derived from earthquake ground motions have been reported recently by Newmark (Ref. 4-12) and by Blume (Ref. 4-13). From the results of these two studies, a unified procedure was developed for defining response spectra, based on a mean-plus-one standard deviation (MSD)* statistical level from all spectra examined. This procedure was subsequently presented in a joint paper by Newmark, Blume, and Kapur (Ref. 4-5), and has since been condensed and presented as Regulatory Guide 1.60 (Ref. 4-1). These references should be consulted for additional background information.

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*Hereafter in this chapter, spectra representing the mean-plus-one standard deviation probability level will be referred to as the MSD spectra.

The Regulatory Guide 1.60 spectra utilize nearly the same general scaling relationships as the Newmark-Hall spectra, but are based upon a greater number of earthquake ground motion records. To define horizontal response spectra, the maximum horizontal ground acceleration and displacement must first be established. The maximum horizontal ground displacement is considered to be proportional to the maximum ground acceleration and is set at 36 in. for a maximum ground acceleration of 1.0g. These values, it should be noted, are proportional to the 18-in. maximum ground displacement and 0.5g maximum ground acceleration established for the "standard" earthquake in the Newmark and Hall procedure. The response spectra bounds are established by five straight lines, similar to the Newmark-Hall method, but the control points are designated by letters A, B, C, and D, and have definite assigned frequencies for all horizontal spectra. Amplification factors for constructing the response spectra are again a function of the percent of critical damping, and are specified in Figure 4-4a for the four control frequencies. Response spectra for 0.1g maximum horizontal ground acceleration have been constructed according to Regulatory Guide 1.60, and are provided in Figure 4-4b.

Vertical response spectra are constructed in the same manner as horizontal response spectra; however, the frequency for control point C is at 3.5 Hz rather than at 2.5 Hz, and the amplification factors are different (Fig. 4-5a). Also, the maximum *horizontal* components of displacement and acceleration are used as scaling factors in constructing the vertical response spectra (Ref. 4-1). The amplification factors in Figure 4-5a have been established so that the resulting vertical response spectra will be two-thirds of the values computed for the horizontal spectra, at frequencies below 0.25 Hz. In the range of frequencies from 0.25 Hz to 3.5 Hz, the ratio of vertical to horizontal spectral amplitudes varies from two-thirds to one; whereas for frequencies between 3.5 Hz and 33 Hz, the horizontal and vertical response spectra are identical. Beyond 33 Hz, the vertical spectral

Percent of Critical Damping	Amplification Factors for Control Points			
	Acceleration ¹			Displacement ¹
	A(33 Hz)	B(9 Hz)	C(2.5 Hz)	D(0.25 Hz)
0.5	1.0	4.96	5.95	3.20
2.0	1.0	3.54	4.25	2.50
5.0	1.0	2.61	3.13	2.05
7.0	1.0	2.27	2.72	1.88
10.0	1.0	1.90	2.28	1.70

¹Maximum ground displacement is taken proportional to maximum ground acceleration, and is 36 in. for ground acceleration of 1.0 gravity.

(a) AMPLIFICATION FACTORS

(b) SPECTRUM SHAPES

FIGURE 4-4. REGULATORY GUIDE 1.60 RESPONSE SPECTRA--HORIZONTAL GROUND MOTIONS (REFS. 4-1, 4-5)

Percent of Critical Damping	Amplification Factors for Control Points				
	Acceleration ¹				Displacement ¹ D(0.25 Hz)
	(A')(50 Hz)	A(33 Hz)	B(9 Hz)	C(3.5 Hz)	
0.5	0.67	1.0	4.96	5.67	2.13
2.0	0.67	1.0	3.54	4.05	1.67
5.0	0.67	1.0	2.61	2.98	1.37
7.0	0.67	1.0	2.27	2.59	1.25
10.0	0.67	1.0	1.90	2.17	1.13

¹Maximum ground displacement is taken proportional to maximum ground acceleration and is 36 in. for ground acceleration of 1.0 gravity.

(a) AMPLIFICATION FACTORS

(b) SPECTRUM SHAPES

(c) COMPARISONS OF HORIZONTAL AND VERTICAL SPECTRA

FIGURE 4-5. REGULATORY GUIDE 1.60 RESPONSE SPECTRA--VERTICAL GROUND MOTIONS (REFS. 4-1, 4-5)

accelerations decrease from a value equal to the peak horizontal ground acceleration at 33 Hz to two-thirds of the horizontal acceleration at 50 Hz (Ref. 4-14).^{*} The resulting family of vertical response spectra are shown in Figure 4-5b and a comparison of horizontal and vertical spectrum shapes for 5% of critical damping is shown in Figure 4-5c.

The spectra presented in Regulatory Guide 1.60 represent the mean-plus-one standard deviation probability level, or the 84th percentile, of a log normal distribution and, as indicated above, are based on the independent studies of Newmark and Blume (Refs. 4-12, 4-13). Newmark's results are derived from 28 records of horizontal ground motion from 9 separate earthquakes, and are in substantial agreement with spectra derived by Blume from a different set of 33 records from 12 earthquakes. Since the two ensembles share 12 records from 4 earthquakes, as shown in Table 4-1, this agreement is not surprising.

4.1.4 COMPARISONS OF SPECTRA

Response spectra developed by Housner, Newmark-Hall, and Regulatory Guide 1.60 procedures are compared in Figure 4-6. This figure indicates relatively small differences between spectra developed by Newmark-Hall and those produced in Regulatory Guide 1.60; these small differences arise primarily from the greater number of records and more recent records considered by Newmark and Blume in their development of the Regulatory Guide 1.60 spectra (Table 4-1). However, the Housner spectra are substantially different from the others; some possible causes for these differences are discussed in the paragraphs that follow.

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^{*}This specification of the vertical spectral acceleration amplitudes above 33 Hz represents a correction to that currently indicated in Revision 1 of Regulatory Guide 1.60. As pointed out by Valera (Ref. 4-14), this correction results in a vertical spectrum shape that is consistent with the original Newmark-Blume-Kapur paper.

FIGURE 4-6. COMPARISONS OF SITE-INDEPENDENT SPECTRA (REFS 4-1 AND 4-3 TO 4-5)

Differences between the Housner spectra and the Regulatory Guide 1.60 spectra must be related primarily to the probability level associated with each approach. The Housner spectra represent an *average* from strong motion records; i.e., these spectra will normally not be exceeded 50% of the time. In contrast to this, the Newmark-Hall and Regulatory Guide 1.60 spectra should *not* be exceeded about 84% of the time, assuming the maximum ground motions have been correctly predicted. The diversities in the probability levels used by Housner and in Regulatory Guide 1.60 can be attributed, in part, to differences in basic design approach at the times these two spectra were developed. When the Housner spectra were developed (during the late 1950's) standard design practice utilized working stress concepts with allowable stresses one-third less than yield. In contrast to this, the current design practice used in conjunction with the Regulatory Guide 1.60 spectrum shape is often based on ultimate strength concepts (particularly with regard to the Safe Shutdown Earthquake as defined in Chapter 3); these concepts are based on allowing the structure to yield under loads 50% greater than for working stress design. Had the Housner spectra also been based on ultimate strength concepts, their spectral amplitudes would conceivably have been increased by 50% and would have compared more closely to Regulatory Guide 1.60, particularly over the 2-to-10 Hz frequency range, the range of greatest interest in nuclear plant design (Fig. 4-6).

In addition to differences in probability levels, variations between the Housner and Regulatory Guide 1.60 spectra can also be attributed to several other factors associated with the records and procedures used in the development of each set of results. These include differences in scaling procedures, numbers of records used, site subsurface characteristics, and ranges in earthquake magnitudes and distances from the causative fault. For example, it has been previously noted that Housner used eight records from earthquakes that vary in magnitude from 6.5 to 7.7, and scaled the spectra using factors based on spectrum intensities. Epicentral distances for these records range from about 10 to 30 mi. In contrast, the Regulatory Guide 1.60 spectra were developed from a much larger sample of strong

motion records, and were based on different procedures for normalizing the individual spectra. Furthermore, the records considered in the development of Regulatory Guide 1.60 represent a wider range of magnitudes and epicentral distances.

Regarding potential effects of variations in subsurface soil conditions, it was previously noted that the records considered by Housner are representative of a variety of soil and rock site conditions. Similarly, Table 4-1 indicates that approximately one-quarter of the records considered in the development of the Regulatory Guide 1.60 spectra were recorded on *rock* sites, several were recorded on *deep cohesionless soil* sites, and the remaining records were obtained on *stiff soil* sites. It should be noted that both Newmark and Blume, in the course of their independent investigations that led to Regulatory Guide 1.60, considered potential effects of local soil conditions; in fact, the Newmark study provided spectra that indicated important differences between motions recorded on sediments and on competent rock, particularly for frequencies below about 3 Hz (Refs. 4-12, 4-13). However, the recommended design spectra contained in the subsequent paper by Newmark, Blume, and Kapur and in Regulatory Guide 1.60 do not differentiate between various local soil profiles, and therefore represent a statistical MSD in which the subsurface soil conditions are an uncontrolled variable.

4.2 SITE-MATCHED RECORDS

4.2.1 GENERAL DISCUSSION

It is clear that the current library of strong motion records, although limited, represents a valuable source of data for use in defining seismic input criteria for nuclear plants. These records can be used to obtain composite site-independent spectra based on records that represent a wide variety of geologic and local site conditions (Sec. 4.1). However, it may also be possible, by careful selection, to identify an ensemble of

spectra from records obtained at stations whose seismologic, geologic, and local soil characteristics more nearly represent those of the proposed plant site. Ensemble mean and MSD spectra could then be obtained using procedures followed by Newmark, Blume, and Kapur in their development of the Regulatory Guide 1.60 spectrum shapes. Spectra selected in this manner would be more consistent with the requirements of the seismic and geologic siting criteria, as defined in Appendix A, 10 CFR, Part 100.

When selecting site-matched records, it is necessary to consider that earthquake ground motions are affected by a number of factors related to the source mechanism, source-site transmission path, and local site conditions. Ideally, records should be selected in which each of these factors nearly matches those of the nuclear plant site and its designated design earthquake events. However, the present limitations in the size of the available library of strong motion records prevents this, thereby necessitating the relaxation of some of these site-matching requirements. Judgment therefore becomes an important factor in first defining those particular factors that might have the greatest effect on the ground shaking at the site, and then selecting those available records that best represent these factors.

4.2.2 ILLUSTRATIVE APPLICATIONS OF SITE-MATCHED RECORDS

4.2.2.1 Guzman-Jennings Approach

In a recent publication (Ref. 4-15), Guzman and Jennings describe a procedure for developing criteria spectra based on site-matched records. They recommend that, after the geological and seismological features of each postulated SSE have been established, an ensemble of strong motion records should be selected that represents comparable conditions of magnitude, distance, local site geology and soil conditions, tectonic province, and fault mechanism. Accelerograms are added to the ensemble until "the conditions under which they were recorded are no longer representative of the SSE, or until records already included from the same earthquake are so clearly more

representative that there is little confidence that additional records are meaningful to the selection of the design spectra." Once this ensemble is selected, existing empirical attenuation curves relating peak acceleration, magnitude, and distance are used to adjust the amplitudes of the selected accelerograms to correspond more closely to the postulated SSE. These attenuation curves are not used to predict peak acceleration; rather they are used to measure the severity of the measured ground shaking *relative* to that expected under conditions corresponding to the SSE. Spectra from the records scaled in this manner are plotted together on a single figure and compared to the Regulatory Guide 1.60 spectrum shape. Based on this comparison an "appropriately conservative set of spectra for the SSE is selected."

Specific example applications of this approach are presented in Reference 4-15. In some of these applications, the Regulatory Guide 1.60 spectrum shape was judged adequate for representing the vibratory motion criteria while, in other applications, criteria spectrum shapes were developed that differed substantially from Regulatory Guide 1.60. This depends on the particular seismologic, geologic, and local soil conditions for the site under investigation, and on the use of sound engineering judgment when selecting or rejecting particular records for inclusion in the ensemble.

4.2.2.2 Seed-Ugas-Lysmer Spectrum Shapes

A comprehensive study of site-dependent spectra based specifically on local site conditions has recently been completed by Seed, Ugas, and Lysmer (Ref. 4-8). In this, ensemble average and MSD spectra were developed from 104 site-matched records corresponding to the four broad site classifications described in Table 4-1 -- rock (28 records), stiff soil (31 records), deep cohesionless soil (30 records), and soft-to-medium soil deposits (15 records). The spectra developed for each site condition corresponded to a damping ratio of 0.05 and were normalized to a zero-period acceleration of 0.1g.

Comparisons of the ensemble average and MSD spectra developed by Seed, Ugas, and Lysmer (Fig. 4-7) indicate trends that can be attributed to differences in local site conditions. For example, judgment would indicate

(a) ENSEMBLE AVERAGE

(b) ENSEMBLE MSD

FIGURE 4-7. SEED-UGAS-LYSMER SPECTRUM SHAPES BASED ON LOCAL SOIL CONDITIONS (REF. 4-8)

that, in the low-frequency range, the spectra from soft-to-medium soil sites and deep cohesionless soil sites should have larger amplitudes than the spectra from rock sites and stiff soil sites; these trends are clearly evident in Figure 4-7 for frequencies of less than 2 to 3 Hz. Furthermore, for higher-frequency responses (above about 6 Hz), spectra from rock sites and stiff soil sites should exhibit larger amplitudes than spectra from the deeper or softer site conditions; these trends are also evident in Figure 4-7, although they are much less pronounced than the trends indicated above for low-frequency responses.

Also of interest in Figure 4-7 are the comparisons of the MSD site-dependent spectra and the Regulatory Guide 1.60 spectrum shape. These comparisons indicate that differences between the site-dependent spectra and Regulatory Guide 1.60 are most pronounced for frequencies below 2 to 3 Hz; less significant but still noticeable differences are noted at higher frequencies.*

4.2.3 VARIABILITY OF INDIVIDUAL MEMBERS OF ENSEMBLE OF SITE-MATCHED RECORDS

As previously noted, the ensemble of site-matched records is selected to correspond to those particular records that best represent the seismologic, geologic, and local soil characteristics of the site. In practice, however, only certain combinations of these particular site characteristics can even be represented by any one record; in fact, the overall set of characteristics represented by each record in the ensemble will invariably differ from one record to another. As a result, the spectrum shapes of the individual records will also differ over frequency ranges of interest in nuclear plant design.

To illustrate, spectra categorized according to local soil conditions are compared in Figures 4-8a and 4-8b. These comparisons indicate that

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*For rock, stiff soil, or deep cohesionless soil sites, higher frequency components of composite spectra differ from Regulatory Guide 1.60 by as much as 25%. Higher frequency components of spectra from soft-to-medium soil sites exhibit larger differences relative to Regulatory Guide 1.60; however, sites of this type are usually not considered to be an appropriate location for a nuclear plant.

STATION AND YEAR	COMPONENT	RICHTER MAGNITUDE	EPICENTRAL DISTANCE MILES	PEAK ACCELERATION G	TYPE OF FAULTING
— PACOIMA DAM, 516E	71	516E	6.5	1.171	REVERSE
--- TAFT	52	569E	7.7	0.178	REVERSE
--- GOLDEN GATE	57	580E	5.3	0.105	STRIKE-SLIP
--- HELENA	35	NS	6.0	0.146	--

*FAULT PASSES BENEATH STATION
**FOCAL DEPTH EQUAL TO 25 MILES

COMPONENTS WITH HIGHER PEAK ACCELERATIONS

STATION AND YEAR	COMPONENT	RICHTER MAGNITUDE	EPICENTRAL DISTANCE MILES	PEAK ACCELERATION G	TYPE OF FAULTING
— PACOIMA DAM	71	S74W	6.5	1.076	REVERSE
--- TAFT	52	N21E	7.7	0.156	REVERSE
--- GOLDEN GATE	57	N10E	5.3	0.083	STRIKE-SLIP
--- HELENA	35	EW	6.0	0.145	--

*FAULT PASSES BENEATH STATION
**FOCAL DEPTH EQUAL TO 25 MILES

COMPONENTS WITH LOWER PEAK ACCELERATIONS

(a) ROCK SITES

STATION AND YEAR	COMPONENT	RICHTER MAGNITUDE	EPICENTRAL DISTANCE MILES	PEAK ACCELERATION G	TYPE OF FAULTING
— EUREKA	54	N79E	6.5	0.257	STRIKE-SLIP
--- ORION	71	NS	6.5	0.255	REVERSE
--- FERNDALE	51	N46W	5.8	0.110	STRIKE-SLIP
--- OLYMPIA	49	N86E	7.1	0.280	NORMAL (?)

COMPONENTS WITH HIGHER PEAK ACCELERATIONS

STATION AND YEAR	COMPONENT	RICHTER MAGNITUDE	EPICENTRAL DISTANCE MILES	PEAK ACCELERATION G	TYPE OF FAULTING
— EUREKA	54	S11E	6.5	0.167	STRIKE-SLIP
--- ORION	71	EW	6.5	0.134	REVERSE
--- FERNDALE	51	S44W	5.8	0.110	STRIKE-SLIP
--- OLYMPIA	49	N04W	7.1	0.160	NORMAL (?)

COMPONENTS WITH LOWER PEAK ACCELERATIONS

(b) DEEP-COHESIONLESS SOIL SITES

FIGURE 4-8. COMPARISONS OF INDIVIDUAL SPECTRA WITH SIMILAR SITE CONDITIONS

individual spectra within the ensemble for each site condition exhibit considerable difference; in fact, in some cases there is overlap between spectra grouped in different site conditions. These differences can be attributed in part to variations in local soil or rock conditions at the individual sites but are probably most directly caused by variations in the source mechanism and source-site transmission path among the various records.

This discussion indicates that, when developing representative spectra according to the site-matching procedures outlined in this section, it is important to utilize an ensemble that contains a sufficient number of records to permit meaningful statistical evaluations of the data. An ensemble of insufficient size could conceivably provide misleading results and trends not entirely representative of the nuclear plant site characteristics. In addition, careful judgment must be exercised when selecting the individual records in the ensemble, so that the entire set of site-matched records provides a reasonable representation of the ground shaking at the site.

4.3 SOIL RESPONSE ANALYSES

It is evident that the use of site-matched records to develop seismic input criteria spectra at a nuclear plant site will define ground motions more nearly representative of the seismologic, geologic, and local soil conditions at the site than will the site-independent procedures indicated in Regulatory Guide 1.60. However, the local soil characteristics at the particular site being evaluated, together with the magnitudes and causative fault distances associated with the designated criteria earthquake events for the site, may not be fully reflected by the currently available library of strong motion records. Furthermore, at sites of many strong motion records, data that define subsurface conditions are often quite limited; therefore, in many cases, these records cannot be applied with complete certainty to a particular site (Sec. 2.3.4). For these reasons, analytical evaluations of the site response take on added importance and may be used to supplement the data from existing strong motion records.

The purpose of this section is to describe current technology for carrying out soil response analyses. Basically, these analyses can be broken down into the three main steps shown in Figure 4-9, namely, the selection of an ensemble of representative subsurface input motions, the development of a mathematical model of the site soil deposits, and the evaluation of the various types of output from the analyses. Each of these steps is discussed in the remainder of this section.

4.3.1 SUBSURFACE INPUT MOTIONS

As discussed previously, the factors that might affect ground shaking at a site include the source mechanism, source-site transmission path, and local soil conditions at the site. However, current technology for analyzing the effects of various source mechanisms and transmission paths is not yet advanced to the point that these effects can be directly considered in engineering applications. As a result, such factors must be represented by an ensemble of time histories that corresponds to the full range of shaking effects that can occur at the base of the soil deposit. This ensemble can consist of surface rock motion records (Sec. 4.2), artificial earthquake records (Sec. 2.3.2), band-limited white noise (Ref. 4-16), or subsurface motions computed from existing ground surface records (Ref. 4-17). Techniques and data for scaling these subsurface input motions are provided in Section 2.3.3 of Chapter 2.

The above paragraph indicates that only the local soil deposits are modeled mathematically in current engineering practice. The considerable complexities associated with the source mechanism and source-site transmission path are represented solely by an ensemble of input motions applied to the soil model. Therefore the importance of providing a reasonable set of subsurface input motions cannot be overemphasized, since for many sites these input motions have a strong effect on the computed soil response near the ground surface.

FIGURE 4-9. SOIL RESPONSE ANALYSES FOR NUCLEAR POWER PLANT SITES

As indicated subsequently in Section 4.3.2, the mathematical models of soil deposits typically extend from the ground surface to subsurface rock like materials.* Input motions are applied at the interface of these rock-like materials and the overlying soil deposits. In the absence of subsurface rock motion measurements, these input motions must be based on either (1) rock outcrop records applied directly at the subsurface rock level; or (2) the use of deconvolution procedures to compute subsurface rock motions from existing surface motions recorded on rock outcropping or on soil deposits.† The surface records used in either of these alternative procedures are scaled to correspond to a given magnitude and distance, based on empirical data.

Perhaps the best available guide to the characteristics of subsurface input motions is the current library of rock outcrop records. The extreme variability of such records has been investigated by Seed and Schnabel (Ref. 4-19), who used calculations and recorded rock motions from the 1971 San Fernando earthquake to show that peak rock accelerations recorded at a given distance from the earthquake source vary by factors of 100% or more; in fact, it was shown that peak acceleration differences of this magnitude may occur between two horizontal components recorded at any one station (Fig. 4-10). Although not specifically investigated in Reference 4-19, similar variations may be expected to occur with regard to the frequency characteristics of rock outcrop records, and both the amplitude and frequency contents of the corresponding subsurface rock motions.

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*Typically, rock-like materials are considered to be those having shear-wave velocities of about 2500 fps or more.

†Deconvolution procedures are most often applied in conjunction with one-dimensional analyses of vertically propagating shear waves (Ref. 4-17), although there have been isolated applications of such procedures to more complex two- and three-dimensional wave propagation problems (Ref. 4-18). With regard to one-dimensional applications, Schnabel, et al. (Ref. 4-17) note that small errors in surface motions or in subsurface material properties tend to be magnified when computing subsurface motions using these procedures. Furthermore, the one-dimensional procedures described in Reference 4-17 are valid only if the primary motions within the site are due to vertically propagating shear waves. Therefore, careful judgment must be exercised when defining input parameters and interpreting results from such calculations.

(a) RANGE OF MAXIMUM HORIZONTAL ACCELERATIONS RECORDED ON ROCK DURING 1971 SAN FERNANDO EARTHQUAKE (REF. 4-19)

(b) INFLUENCE OF BASE-ROCK CHARACTERISTICS ON RESPONSE SPECTRA OF GROUND-SURFACE MOTIONS FOR 300-FT SAND DEPOSIT (REF. 4-23)

FIGURE 4-10. SUBSURFACE INPUT MOTIONS FOR SOIL RESPONSE ANALYSES

There are a number of possible causes for this extreme variability in the rock motion records. First, the direction of movement along the fault relative to the site location can be important, especially where reverse or normal faulting is involved. Second, local geologic structures may strongly influence the transmission characteristics of the seismic waves reaching the recording station. This was illustrated by Lysmer and Drake, who used finite element procedures to study Love-wave propagation along a rock formation with a structural discontinuity represented by a shale-filled valley (Ref. 4-20); their study indicated marked changes in amplitudes and irregular amplification of the motions along the surface of the shale.

A third source of the variability of rock motion records is associated with local rock properties and topography. For example, studies of rock motions measured during Japanese earthquakes have indicated peak accelerations on locally weathered and cracked rock to be considerably higher than on sound bedrock (Ref. 4-21). The potential effects of local topography are evidenced by the apparent influence of the surrounding terrain on the high accelerations recorded at Pacoima Dam during the 1971 San Fernando earthquake. The particular location of the accelerograph at Pacoima Dam, on a narrow rocky spine adjacent to the dam abutment, is also judged to have had an important effect on these rock motion records (Ref. 4-22).

The material characteristics of the rock formations themselves represent a fourth possible cause of variation in rock motion characteristics. The effects of material properties of the underlying rock on the ground surface response of various soil deposits was studied by Lysmer, et al., (Ref. 4-23) and some typical results are summarized in Figure 4-10b. In this, Lysmer, et al. showed that, for many of the cases considered, the ground surface shaking characteristics were relatively insensitive to variations in rock properties. However, in some cases, such as those shown in Figure 4-10b where bedrock consists of 500 ft of shale over a hard-rock or rigid base, variations in rock properties resulted in marked differences in ground surface response spectra over frequency ranges important to nuclear plant structures.

4.3.2 MATHEMATICAL MODELING OF SOIL DEPOSITS

The mathematical model of the soil deposit must represent the manner in which the seismic waves are being transmitted through the soil to the ground surface. It must incorporate an appropriate representation of the strain-dependent dynamic properties of the soil materials and, in addition, must properly consider the geometry of the soil deposit. When the bedrock has a horizontal surface of great extent and the overlying soil layers are also horizontal, the response can be attributed primarily to shear waves propagating vertically through the soil from the underlying bedrock. These conditions lead to certain simplifications in the mathematical modeling of the site. However, when the soil layers or underlying rock strata are irregular or sloping, the propagation of seismic waves is more complex, and more elaborate mathematical representations of the site must therefore be used. A summary of the mathematical models used to represent soil deposits is provided in Table 4-2.

The remainder of this section provides additional information about the modeling of soil deposits. Material property models most widely used in current practice are first described. A discussion of various approaches for representing deposits with horizontal boundaries is then provided, followed by a description of techniques for modeling soil deposits with irregular or sloping boundaries.

4.3.2.1 Soil Material Property Representations

To properly conduct a soil response analysis, it is important to recognize the strain dependence of the stiffness and energy dissipation characteristics of soil materials, as illustrated in Figure 4-11. This section contains a general description of some of the models used in current practice to represent the strain-dependent nature of soil materials. Those particular properties required in one-dimensional and two-dimensional models are described in Table 4-2 and in Section 4.3.2.2.

TABLE 4-2. SUMMARY OF MATHEMATICAL MODELS USED FOR SOIL RESPONSE ANALYSIS

Site Condition	Type of Model	Required Soil Properties	Comments
Essentially Horizontal Boundaries	One-Dimensional Continuum Models	Unit weight, shear modulus, and damping ratio. Shear-wave velocity may be used to estimate shear modulus at small strains, while strain dependence of shear modulus and damping factor may be accounted for through equivalent linear models. Relative density of sands and shear strength of clays may be used to obtain properties from curves of the type shown in Fig. 4-12(b). Viscous damping, because of its mathematical simplicity, is most often used to represent hysteretic energy losses in soil materials	Closed-form solutions are available for the response of linear soil models with discrete layers or for a class of models with continuously varying properties if excitation is simple. Numerical transform techniques allow the recovery of ground motion histories under more realistic excitation. Iterative procedure may be used with strain-dependent equivalent linear models.
	One-Dimensional Discrete Mass Models		Modal superposition solutions used with iterative procedure allow treatment of strain-dependent properties. Discrete elements or layers provide variation of modulus with depth; some limitations on variation of damping unless direct integration solution is adopted.
Irregular or Sloping Boundaries	Two-Dimensional Finite Element Models	Shear modulus, bulk modulus, and damping factor (as functions of current strain, previous history and loading condition if nonlinear models are used). Unit weight and Poisson's ratio are also required.	Can accommodate changes in soil properties in two directions (horizontal layering not required) and complex boundary geometry. Treats both shear waves and P-waves. Modal superposition and iteration to account for strain dependence are possible. Direct integration with nonlinear soil models provides alternative means of accounting for strain dependence. Two-dimensional models require significantly longer computing times than do one-dimensional models.

FIGURE 4-11. EFFECT OF STRAIN LEVEL ON SHEAR MODULUS AND DAMPING
(REF. 4-24)

Perhaps the most widely used and simplest means of including these characteristics is the equivalent linear model developed by Seed and Idriss (Ref. 4-24). In this, the soil is represented as a linear viscoelastic material whose stiffness and energy dissipation characteristics are introduced by using an equivalent shear modulus and an equivalent viscous damping coefficient. Each of these parameters can vary with the depth of the soil layer and the strain level within the layer. As shown in Figure 4-12a, the equivalent shear modulus is taken as the slope of the diagonal line drawn through the ends of the hysteresis loop, whereas the average equivalent viscous damping coefficient is assumed proportional to the area of the hysteresis loop. Curves such as those shown in Figure 4-12b have been developed for a number of the soil materials and can be used to evaluate these equivalent parameters as a function of strain level.

The implementation of the equivalent linear approach in the soil response analysis is typically an iterative process. In this, soil moduli and damping values are first estimated for each soil element in the model. Then the system is analyzed using these properties, and acceleration and strain time histories are computed throughout the soil deposit. From these time histories, effective soil-strain amplitudes are estimated in each soil element, and curves similar to Figure 4-12b are consulted to see if the moduli and damping values used in the response evaluation are compatible with the strains developed. If the soil properties are not compatible, these curves are used to provide improved values of moduli and damping for the next iteration and the process is repeated until convergence has occurred, usually within three to five iterations. The response from the last iteration is considered to correspond to the nonlinear response.

A second technique that has been used to represent the strain dependence of the soil material properties is a bilinear representation. In this, the actual stress/strain characteristics of the soil material are represented as a series of straight lines of appropriate orientation that

$$(\lambda_{eqh})_{\gamma_1} = \frac{\text{Area of hysteresis loop}}{4\pi \times \text{Area OAB}}$$

$$(\lambda_{eqh})_{\gamma_2} = \frac{\text{Area of hysteresis loop}}{4\pi \times \text{Area OCD}}$$

(a) DETERMINATION OF EQUIVALENT SHEAR MODULUS, G_{eq} , AND DAMPING RATIO, λ_{eqh}

(b) TYPICAL CURVES USED TO DEFINE STRAIN-DEPENDENT PROPERTIES

FIGURE 4-12. EQUIVALENT LINEAR METHOD (REF. 4-24)

are so interconnected as to best represent the actual soil properties (Fig. 4-13a). Hysteretic damping of the soil material is represented independently of the bilinear model. For example, the bilinear lumped mass model developed by Parmelee, et al. (Ref. 4-25), utilizes a series of dashpots, one representing hysteretic damping and the other representing creep* (Fig. 4-13b).

It is noted that the implementation of the bilinear model requires storage of the previously experienced maximum and minimum strain amplitudes and, in addition, must incorporate tests at selected real-time intervals to determine whether the material is loading, unloading, or reloading. This representation is obviously more complex and time consuming than the equivalent linear model; however, results from comparable analyses using both soil models have indicated only minor differences, as shown in Figure 4-13c (Ref. 4-26). Therefore, it would appear that because of its simplicity the equivalent linear representation is preferable.

Other more complex representations of soil material behavior have been developed, primarily for use in finite element or finite difference programs (Refs. 4-27, 4-28). These models can represent the material properties as a function of stress, strain, stress and strain histories, or strain rate, and may include appropriate yield criteria and flow rules based on the theory of plasticity. The primary advantage of these complex models is that they provide a more complete and theoretically correct representation of soil or rock material behavior. However, the use of these models is extremely time consuming, and furthermore it may not be possible within the limits of current soil sampling and testing technology to develop sufficiently

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*Subsequent laboratory tests and computations carried out by Parmelee, et al. in conjunction with this study indicated that the soil responses would be unaffected by creep. Therefore, the creep dashpots were considered locked when performing soil response analyses using this model.

(a) HYSTERETIC AND EQUIVALENT BILINEAR REPRESENTATION FOR SOIL MATERIALS

(b) EXAMPLE REPRESENTATION OF SOIL DAMPING IN BILINEAR MODEL (REF. 4-25)

RESPONSE SPECTRA

(c) COMPARISON OF RESULTS OF SEISMIC ANALYSIS OF 30-FT CLAY LAYER USING BILINEAR AND EQUIVALENT LINEAR METHODS (REF. 4-26)

Depth, in feet	Maximum Shear Stress, in pounds per square foot	
	Bilinear solution	Equivalent linear solution
1	31	26
3	86	78
5	145	131
7	172	167
9	245	215
11	273	251
13	297	283
15	301	309

MAXIMUM SHEAR STRESSES

FIGURE 4-13. BILINEAR REPRESENTATION OF SOIL MATERIAL PROPERTIES

accurate parameters to define these models. It would certainly seem that additional study to improve field sampling procedures (to reduce the degree of disturbance of the soil samples) and testing procedures (to provide appropriate parameters for these more refined models) is required so that the application of these refined soil material representations can be justified. However, until this occurs, the equivalent linear approach seems most appropriate for application to soil deposit response analyses.

4.3.2.2 Representation of Deposits with Essentially Horizontal Boundaries

Many potential sites of nuclear power plants can be represented as a series of horizontal, parallel layers of soil extending horizontally for a distance several times larger than their total depth and overlying a horizontally extending surface of bedrock material. For this condition, the horizontal ground motions within the soil deposits arise from vertically incident shear waves that propagate from the underlying rock formations. The soil layers can then be treated as semi-infinite, and a one-dimensional model of the soil deposits extending to rock can be used to predict the horizontal ground response. For these simplified models, only the layer depths, shear moduli, damping, and mass densities need be defined.

Two types of one-dimensional models have been used to compute the response of soil deposits. The first involves use of the one-dimensional continuum model (Fig. 4-14a); this approach was first applied by Japanese investigators in the 1930s (Refs. 4-29, 4-30) and since has been extended by a number of investigators including Kanai (Ref. 4-31) and, in the United States, Seed, Whitman, and their associates (Refs. 4-32, 4-33). In this approach, the strain dependence of the soil material properties is represented through the equivalent linear model described in Section 4.3.2.1. The resulting analysis is generally carried out in the frequency domain, by utilizing the Fourier transform of the subsurface bedrock input motions to represent this input as the superposition of harmonic signals of different frequencies. The frequency-dependent transfer function for the soil deposit

(a) ONE-DIMENSIONAL CONTINUUM REPRESENTATION OF SOIL DEPOSIT (REFS. 4-29 TO 4-33)

(b) DISCRETE MASS REPRESENTATION OF j TH SUBLAYER IN SOIL DEPOSIT (REF. 4-24)

FIGURE 4-14. MATHEMATICAL REPRESENTATIONS OF DEPOSITS WITH HORIZONTAL BOUNDARIES

is obtained by using the one-dimensional wave equation to compute the response of the deposit to a number of harmonic base motions of unit amplitude and different frequencies. The time-dependent response of the deposit to the input bedrock motions is then obtained as the inverse transform of the product of the various harmonic signals comprising the base excitation and the transfer function of the soil deposit.

The second type of one-dimensional model involves the representation of the soil deposit as a series of arbitrarily spaced discrete masses interconnected by shear springs and viscous dampers. In this, the shear spring stiffnesses are dependent on the shear modulus and the spacing of the discrete masses. The strain dependence of the shear modulus and resulting spring stiffness can be represented according to the equivalent linear model, bilinear model, or other appropriate nonlinear soil representation, although the equivalent linear model is currently most widely used. Likewise, the hysteretic damping coefficients can be specified separately for each soil material in the profile. Typically, these damping characteristics are represented by one or more viscous dashpots, as shown in Figure 4-14b (Ref. 4-24). Once these stiffness and damping characteristics for each soil material are defined, the mass, stiffness, and damping matrices for the entire soil deposit can be formed. The resulting matrix equations of motion can then be solved by conventional dynamic analysis procedures. This solution technique may be iterative, if an equivalent linear soil model is used, or may involve a time-marching solution technique if a more detailed nonlinear soil model is implemented.

A number of comparisons of results obtained using the continuum and discrete-mass models have indicated that both models are essentially equivalent and are equally applicable for performing soil response analyses. For example, Roesset and Whitman used both models to compute amplification curves for soil profiles subjected to periodic base motions as well as non-periodic motions (Ref. 4-33). They conclude that for either case the

continuum and discrete-mass models are equivalent, if appropriate judgment is exercised in selecting the values of the parameters for each model. In addition, Tsai and Housner have indicated a close correlation of results obtained using the continuum and discrete-mass approaches for soil deposits on either a flexible rock or firm rock base (Ref. 4-34).

Both the continuum and discrete-mass models have been applied at sites where strong motion records were obtained, and have produced results that agree well with recorded data. Schnabel, et al., have applied a one-dimensional continuum approach with an equivalent linear soil model to sites where motions were recorded during the 1957 San Francisco earthquake (Ref. 4-17). Their results are summarized in Figure 4-15 and show a reasonable correlation with measurements. Seed has applied the discrete-mass approach with an equivalent linear soil model to a number of strong motion accelerograph sites and has also obtained reasonable agreement with recorded motions, as shown in Figure 4-16 (Ref. 4-35).

When applying one-dimensional techniques, it should be realized that typical soil deposits will deviate, sometimes substantially, from the ideal case of horizontal boundaries and layers; as a result, the response of these soil deposits is often more complicated than simple, vertically propagating shear waves. Despite these complexities, one-dimensional models can often serve to provide adequate engineering estimates of ground motions at a site with reasonably horizontal boundaries -- provided that sound judgment is applied in the modeling procedures and interpretation of results, and that certain inherent limitations in the one-dimensional approach are recognized. The more important of these limitations are as follows:

- a. Since one-dimensional models consider only vertically propagating shear waves, the spectra of ground motions computed from these models may be deficient in frequency ranges representative of other types of seismic waves that may be

(a) RESPONSE OF STATE BUILDING SITE COMPUTED FROM MOTIONS RECORDED AT SOUTHERN PACIFIC BUILDING SITE

(b) COMPARISON OF RESPONSE SPECTRA FOR COMPUTED AND RECORDED MOTIONS

FIGURE 4-15. COMPARISONS OF RECORDED MOTIONS WITH COMPUTATIONS USING ONE-DIMENSIONAL CONTINUUM MODEL (REF. 4-17)

(a) ALAMEDA PARK, MEXICO CITY--1955 MEXICO CITY EARTHQUAKE

(b) ALEXANDER BUILDING--1957 SAN FRANCISCO EARTHQUAKE

FIGURE 4-16. COMPARISONS OF RECORDED MOTIONS WITH COMPUTATIONS USING ONE-DIMENSIONAL DISCRETE MASS MODEL (REF. 4-35)

present. For example, where long-period surface waves might be important, the low-frequency content of the ground motions would be underestimated if one-dimensional techniques were used.

- b. One-dimensional models of the type described above are appropriate only for computing horizontal motions of the soil deposit and do not provide a means for predicting vertical soil motions. Vertical-motion computation using a one-dimensional model that considers vertical propagation of P-waves has been suggested (Ref. 4-33); however, this approach is not yet developed to the point that it can be incorporated into the current technology associated with soil response analyses. As a result, vertical soil motions must be computed using two-dimensional techniques described in the paragraphs that follow, or, alternatively, can be estimated from site-matched records. In view of the important effects of vertical soil motions on nuclear plant equipment responses and floor response spectra, this is one area for which additional study is required.

4.3.2.3 Representation of Deposits with Irregular or Sloping Boundaries

If a soil deposit has irregular or sloping boundaries, it can no longer be treated as semi-infinite, and more complex analytical procedures are required to account for the two-dimensional aspects of the soil response. For this case, current technology is generally restricted to the use of two-dimensional finite element representations of the soil deposit to carry out the seismic response analysis.

In the finite element technique, the soil deposit is idealized as an assemblage of two-dimensional triangular or rectangular elements interconnected at a finite number of nodal points, as shown in Figure 4-17

(a) IDEALIZATION OF EARTH BANK

(b) IDEALIZATION OF SOIL DEPOSIT UNDERLAID BY INCLINED ROCK SLOPE

FIGURE 4-17. FINITE ELEMENT REPRESENTATION FOR TWO-DIMENSIONAL SEISMIC RESPONSE ANALYSIS OF SOIL PROFILES (REF. 4-35)

(Ref. 4-35). Material properties can be specified uniquely for each of the individual elements in the finite element grid so that variations in geometry and layering characteristics can be accommodated. As indicated in Table 4-2, the two-dimensional finite element model requires a more extensive definition of material properties than do one-dimensional models. If an equivalent linear material representation is used, then the shear modulus, bulk modulus or Poisson's ratio, and damping factor as a function of strain amplitude must be defined for each element, in addition to its unit weight. If nonlinear models are used, then these properties may also be a function of strain rate and prior load history or, as an alternative, the hysteretic stress/strain characteristics of each soil material may be defined directly.

The behavior of the finite element system is described by assigning a displacement field to each element such that compatibility of strains is maintained at the interface of adjacent elements. On the basis of this assumption, and utilizing the equivalent linear or nonlinear soil representation prescribed for each element, it is possible to define the equations of motion for the system. In these equations, the vector of internal element forces will be dependent on strain amplitude, and possibly strain rate and prior load history as well, depending on the soil model used. Conventional normal mode or time-marching integration techniques can be implemented to solve these equations of motion for the displacement, velocity, and acceleration of each node point, from which element strains and stresses can be readily obtained.

Finite element methods have been used to investigate the response of embankments (Ref. 4-36), earth banks (Ref. 4-37), and soil deposits underlain by sloping rock surfaces (Ref. 4-38). These investigations have been carried out using a variety of soil material representations ranging from linear viscoelastic (Ref. 4-36) to equivalent linear (Ref. 4-38) to nonlinear models (Ref. 4-39). Example results from finite element calculations have been described by Seed (Ref. 4-35) and are summarized in Figure 4-18.

(a) COMPUTED GROUND MOTIONS-- ANALYSIS OF SLOPING ROCK SURFACE

(b) PEAK ACCELERATIONS AND RESPONSE SPECTRA--ANALYSIS OF EARTH BANK

FIGURE 4-18. EXAMPLE SOIL RESPONSE ANALYSES USING TWO-DIMENSIONAL FINITE ELEMENT TECHNIQUE (REF. 4-35)

The nature of these results indicates the implications of using one-dimensional models in the vicinity of a soil deposit overlying a sloping rock surface (Fig. 4-18a), and depicts the kind of valuable design information that can be obtained from soil response analyses. For example, in Figure 4-18b, an analysis of an earth bank can be used to indicate the variability of peak ground accelerations in the vicinity of the bank, and the fact that these peak accelerations can be somewhat higher just beyond the crest of the slope than at locations well beyond the slope. This figure also depicts the kinds of changes in response spectra that may occur in the vicinity of an earth bank (Ref. 4-35).

To date, no comparisons between measured records and finite element computations of the seismic response of soil deposits are available in the published literature. However, Kovacs, et al., have shown good correlation between finite element analysis results and measurements of the response of small scale embankments subjected to base motions on shaking tables, as shown in Figure 4-19 (Ref. 4-40). In addition, finite element methods generally give results in overall agreement with those computed using one-dimensional models of sites with horizontal boundaries; as indicated previously (Sec. 4.3.2.2), one-dimensional analysis results have shown reasonable agreement with strong motion records. This suggests that finite element techniques are adequate for use in two-dimensional soil response analyses.

4.3.3 OUTPUT FROM SOIL RESPONSE ANALYSES

One of the primary advantages of conducting soil response analyses is the variety of output that can be obtained. Computed soil response data in the form of motions, strains, or stresses can be obtained either in the time or frequency domain and at any location within the soil deposit. Much of this available output data is summarized in Figure 4-9 and in a number of the other figures throughout Section 4.3. It is appropriate here to briefly summarize some of this information.

$$\text{BASE ACCELERATION} = \ddot{x}_0 \sin 2\pi ft$$

$$\ddot{x}_0 = 0.05G$$

$$f = 36 \text{ HZ}$$

FIGURE 4-19. COMPARISON OF EXPERIMENTAL RESULTS WITH FINITE ELEMENT ANALYSIS RESULTS FOR SMALL SCALE EMBANKMENTS (REF. 4-40)

Output data in the time domain that can be obtained from soil response analyses include time histories of ground acceleration, velocity, and displacement at any point within the soil deposit. The variations with depth of these soil motions can be particularly important for nuclear power plant structures that are embedded well below the ground surface, since for many types of subsurface soil conditions, the soil motions at the base of these deeply embedded structures can be substantially different from those at the ground surface. Furthermore, current trends point toward the increased use of finite element soil/structure models for analyzing the dynamic response of nuclear power plant structures (Ref. 4-27); these models often require a definition of the variation of soil-motion time histories along the depth of the soil profile as well as at the base rock level. Therefore, subsurface motion time histories obtained from soil response analyses can be an important guide for defining appropriate seismic input for nuclear plant structure analyses.

A second form of output from the soil response analyses is in the frequency domain and includes response spectra, spectral amplification factors, and Fourier spectra. Current design practice for nuclear plants is often based on the use of site-independent response spectra, which, as discussed in Section 4.1, are based solely on ground surface records. These spectra are typically treated as input to soil/structure interaction springs and dashpots at the base of the structure, even though the structure is typically embedded. Therefore, the above comments regarding the depth dependence of soil-motion time histories apply equally well to response spectra; i.e., it is apparent that soil response analyses can provide guidance for determination of input response spectra that incorporate the depth dependence of the soil motions.

Other possible forms of output in the frequency domain are the spectral amplification factor (SAF) curves and the Fourier spectrum. The SAF curves correspond to a plot of the ratio of the computed soil response

spectrum ordinates to the subsurface input spectrum ordinates, as a function of frequency. Roesset and Whitman have pointed out a number of advantages in using the SAF, such as its simplicity and its ability to represent local site conditions by a single curve* (Ref. 4-33). The Fourier spectrum of the computed soil motions not only shows their significant frequency characteristics, but also can be used directly as input to nuclear plant structure analyses (Ref. 4-41). It is noted that, like the response spectrum and time histories, both the SAF and Fourier spectra can be obtained from the soil response analyses at any location within the profile.

A third form of output from these analyses is the soil stresses and strains. This information is typically required in order to evaluate the stability of the deposit under earthquake conditions. In addition to the peak stresses and strains, the time history of these quantities is also required to determine the number of cycles for which given stress or strain levels are attained.

It is again emphasized that each of the forms of output described above should be defined in a manner that accounts for the various uncertainties associated with carrying out soil deposit analyses. Therefore, appropriate parametric studies are necessary to determine the sensitivity of the site response to these uncertainties, such as those associated with the definition of subsurface input motions and soil properties. Output from these studies should provide an ensemble of results from which statistical and probabilistic inferences about the nature of the soil response can be made. A deterministic analysis of the earthquake-induced response of soil deposits, based on a single calculation or a limited number of calculations, will not account for these uncertainties in an adequate manner.

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*According to Roesset and Whitman, the SAF is typically insensitive to the structural damping used to define the input and output response spectra.

4.4 SUMMARY AND EVALUATION

At this time it is appropriate to briefly summarize current technology for the estimation of earthquake ground motions, as described in the prior portions of this section. Site-independent methods (Sec. 4.1) and site-dependent methods (Secs. 4.2, 4.3) are summarized and evaluated in Table 4-3 and in the following discussion.

4.4.1 SITE-INDEPENDENT METHODS

Site-independent methods typically apply statistical techniques to an ensemble of spectra from measured records to derive standardized spectrum shapes for use at all sites, regardless of local soil conditions. As discussed in Section 4.1, the Housner, Newmark-Hall, and Regulatory Guide 1.60 spectrum shapes have been most widely used to define seismic input criteria in past nuclear plant applications (Refs. 4-1 and 4-3 to 4-5). These various spectrum shapes differ according to the number of records considered in the statistical processing, and the probability levels considered in the definition of the spectrum shapes, (i.e., whether mean or MSD spectra are obtained).

The use of these site-independent spectrum shapes for defining seismic input criteria has a number of important advantages (Table 4-3). Perhaps the most important advantages are the simplicity of the method and the fact that it has been considered to be an acceptable approach (for stiff soil sites) by the USNRC regulatory staff. However, there are also drawbacks to the approach, as discussed in Section 4.1. The most important drawback is that the site-independent methods are based on a limited library of strong motion records. Furthermore, these methods typically do not consider the potential effects of variations in depth to rock or in local soil conditions, which can conceivably have important effects on the resulting ground motion spectra (Sec. 4.2.).

TABLE 4-3. SUMMARY OF TECHNOLOGY FOR ESTIMATING EARTHQUAKE GROUND MOTION CRITERIA

Method	Procedure	Merits	Limitations
Site-Independent Approach Based on Standardized Spectrum Shapes	(1) Use standardized spectrum shapes that have been derived from statistical analyses of spectral characteristics of measured accelerograms, independent of local site conditions. (2) Derive peak acceleration for site from empirical data relating peak acceleration to either intensity or to magnitude and causative fault distance. (3) Scale spectrum shapes from (1) to correspond to peak acceleration levels from (2). (4) To obtain time histories at ground surface, can use Fourier component synthesis approach based on one of the series of damped spectra.	(1) Has been considered to be an acceptable procedure for defining seismic input response spectra for stiff soil sites by USNRC regulatory staff. (2) Simplicity of approach. (3) No special soils data needed.	(1) Limited data base of measured accelerograms used to develop spectrum shapes. (2) Uncertainties in determination of peak acceleration from empirical data. (3) Spectrum shapes may not be compatible with local site conditions. For this case, variations of motion with depth cannot be obtained.
Site-Dependent Approach Based on Site-Matched Records	(1) Collect an ensemble of measured accelerograms measured at stations whose regional geology, tectonics and local soil conditions best correspond to those at site of interest. In addition, magnitude and distance associated with the ground motion records should approximate those defined for site SSE. (2) Use these accelerograms to obtain spectrum shapes that correspond to prescribed statistical level. (3) Derive peak accelerations for site from empirical data relating peak acceleration either to intensity, or to magnitude and causative fault. Where possible, acceleration levels from strong motion data corresponding closest to particular geologic and local soil conditions of nuclear plant site should be used. (4) Scale spectrum shapes from (1) to correspond to peak acceleration criterion defined for site.	(1) Measured records and resulting ensemble average and MSD spectra are selected to represent, as nearly as possible, SSE magnitude, distance and fault mechanism, as well as regional geology, tectonics and local soil conditions. (2) Variations of motions with depth can possibly be estimated from deconvolution procedures.	(1) Limitations in data base of measured accelerograms related to particular site conditions. (2) Site characteristics at locations of recording stations not often known with certainty. (3) Uncertainties in determination of peak acceleration from empirical data.
Site-Dependent Approach Based on Response Analysis of Soil Deposits	(1) Evaluate whether one-dimensional or two-dimensional soil profile model is appropriate for site. (2) Select ensemble of subsurface rock input motions from existing measurements on rock outcrops, artificial earthquakes, band-limited white noise or suitable computations.* (3) Utilize appropriate nonlinear soil parameters in the model, based on field and laboratory investigations of the site profile. (4) Compute ensemble of motion time histories along depth of profile. Ensemble response spectra at ground surface and at foundation level, corresponding to prescribed statistical level, can also be obtained.	(1) Variations of soil motion with depth are examined. (2) Computed motions are compatible with local soil effects. (3) Can be used to perform sensitivity studies of various site parameters.	(1) Limited data on which to base selection of subsurface input motions. (2) Uncertainty in estimating soil material properties required for dynamic analyses, particularly for soil layers at substantial depth and when refined material models are utilized.

*To define strength level of input motions, empirical data should be used that relate peak rock acceleration to magnitude and distance or to other parameters that define significant earthquake events for site.

4.4.2 SITE-DEPENDENT METHODS

To provide seismic input criteria more nearly representative of geologic and local soil conditions at the site, two types of site-dependent methods can be used. The first consists of the use of site-matched records, while the second is based on soil response analyses.

4.4.2.1 Site-Matched Records

This approach for estimating earthquake ground motions utilizes measured records obtained at stations whose regional and local site conditions are similar to those of the proposed nuclear plant site. An ensemble of these site-matched time-history records can be used to derive smoothed spectrum shapes if a response spectrum analysis of the structure is to be used. In addition, since these measured ground motions are reasonably compatible with the local soil conditions, recently developed deconvolution procedures can be used in certain cases to estimate a consistent set of sub-surface soil motions for use as input to nuclear plant structure analyses (Ref. 4-17).

The use of site-matched records, although based on actual ground motion measurements, must be evaluated in terms of the degree to which the magnitude, fault distance, source mechanism, and local soil profile for the recording station matches those conditions at the nuclear plant site. As is the site-independent method, the use of site-matched records is limited by the number of strong motion measurements currently available; therefore it will invariably be necessary to relax several of the above matching requirements in order to obtain a reasonable number of records to comprise the ensemble for a particular site.

4.4.2.2 Soil Response Analyses

The use of analytical models to examine the earthquake-induced response of local soil deposits offers a number of advantages for nuclear plant applications, as indicated in Section 4.3. First, the variation of soil response with depth and location is directly examined; the present strong motion network does not provide a comparable observation base from which to address this problem. Second, ground motions obtained from response analyses are compatible with the soil models adopted. As a result, if these motions are to be used in the dynamic analysis of soil/structure interaction and structural response, they provide a more consistent definition of input motion than those derived from standardized spectrum shapes. The most important advantage, however, is that the use of soil response analysis methods allows parametric studies to be conducted so that the sensitivity of the site response to variations in the subsurface input motions or in the description of the soil profile can be examined.

Soil response analyses must serve to represent the subsurface input motions, soil material properties, and geometry of the deposit in an adequate manner. Subsurface input motions should consist of an ensemble of real, artificial, or computed records that anticipate the full range of shaking effects that could conceivably occur with the subsurface bedrock level at the site. The soil material model used in the analysis must represent the nonlinear behavior of the various soil layers at strain levels representative of the criteria earthquake events for the site. Equivalent linear, bilinear, or more elaborate material models can be used although equivalent linear representations are most widely applied at this time. One-dimensional or two-dimensional models are utilized to represent the geometry of the soil deposits depending on whether the site contains horizontal boundaries or irregular boundaries.

Obviously, the use of soil response analyses to estimate earthquake motions at a nuclear plant site is an extremely complex problem, and requires considerable care and judgment when developing the model and interpreting the results. However, properly conducted and interpreted analyses of this type can serve to provide valuable design information regarding the sub-surface and surface response characteristics of the site.

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CHAPTER 5

DESCRIPTION AND APPLICATION OF PROCEDURE FOR DEFINING SEISMIC INPUT CRITERIA

In this chapter, information from the previous chapters of this report is utilized to define a procedure that includes site-dependent effects as a partial basis for selection of seismic input criteria for nuclear power plant sites. In addition, an example of the procedure is presented to clearly identify the various step-by-step processes involved.

5.1 DESCRIPTION OF PROCEDURE

In Figure 1-1 of Chapter 1, a procedure was outlined that is consistent with the requirements of Appendix A, 10 CFR, Part 100, for selecting seismic input criteria for a nuclear plant site (Ref. 5-1). This procedure consists of the following six steps:

1. Geologic and Seismologic Studies
2. Site Soils Investigations
3. Site Response Sensitivity Studies
4. Evaluation of Local Site Response Characteristics
5. Selection of Site-matched Records
6. Appraisal and Selection of Seismic Input Criteria

This input selection procedure, as described in the remainder of this section, considers the results of soil response analyses, site-matched records, and Regulatory Guide 1.60 standardized spectrum shapes (Ref. 5-2). As a result, all phases of current seismic response technology, weighed by sound engineering judgment regarding the characteristics of the particular site, are incorporated in the final determination of seismic input criteria for the nuclear plant site.

5.1.1 STEP 1: GEOLOGIC AND SEISMOLOGIC STUDIES

The geologic and seismologic studies of the region surrounding the nuclear plant site establish the basis for selection of vibratory ground-motion input criteria for the site. Such studies, together with the site soils investigations (Step 2), also comprise the starting point from which the other site evaluation requirements specified in Reference 5-1 can be considered. These additional requirements, which are not included in the scope of this report, include evaluations of the potential for, and effects of, near-site surface faulting, earthquake-induced floods and water waves, soil liquefaction, slope instabilities, and so forth.

The regional geologic and tectonic investigations required for the nuclear plant site include the determination of the physiography and the lithologic, stratigraphic, and structural geologic conditions and history of the surrounding area. Capable faults and tectonic provinces, as defined in Appendix A, 10 CFR, Part 100, must be identified in the region surrounding the site.

The seismologic studies include the estimation of Richter magnitudes, epicentral intensities, epicentral locations, and rates of attenuation with distance for those earthquakes that have occurred within 200 mi of the site. From a compilation of these data, regional attenuation curves and recurrence interval information can be obtained.

The next phase of Step 1 is the definition of potential SSE events for the site. These events should be identified by moving each epicenter to that point along its causative fault closest to the site. If a particular event cannot be associated with a causative fault, then the epicenter of that event is moved to the point within its tectonic province closest to the site; when the event and the site are within the same tectonic province, the event is assumed to be centered at the site.

Once potential SSE events are identified, empirical data that interrelate important ground shaking characteristics with intensity or with magnitude and distance should be selected. Among the vibratory ground motion characteristics typically estimated from these data are peak rock accelerations, the duration of strong shaking, and peak ground surface accelerations. The peak rock acceleration data form a basis for defining the strength of subsurface input motions used in the site response sensitivity studies of Step 3. The duration of strong shaking data help to establish the length of the subsurface input motions and, in addition, provide guidance in the selection of site-matched records in Step 5. The peak ground accelerations estimated from empirical data serve as first estimates that are used in conjunction with results from soil response analyses to define the peak ground acceleration criteria for the sets (see Step 4). Empirical information of this type is described in Section 2.3.

5.1.2 STEP 2: SITE SOILS INVESTIGATIONS

The site soils investigations consist of a series of field and laboratory tests to determine the geophysical and mechanical properties of the various soils within the site. These investigations serve to provide the basic data for the site response sensitivity studies (Step 3). In addition, the dominant site features indicated by these soils investigations provide a partial basis for the selection of site-matched records in Step 5.

Field and laboratory procedures for determining site conditions and soil properties are described in detail in Reference 5-3 and are summarized in Chapter 3. The field procedures consist of trenching, exploratory borings, and geophysical investigations that serve to establish basic soil types and layer thicknesses, relative densities, P-wave and shear-wave velocities at small strains, groundwater conditions, and depths to bedrock. Laboratory tests of soil samples obtained in the field define static and dynamic engineering soil properties, including the strain-dependent shear moduli and hysteretic damping properties required for the dynamic response analysis.

5.1.3 STEP 3: SITE RESPONSE SENSITIVITY STUDIES

The purpose of the site response sensitivity studies is to establish reasonable bounds on the dynamic response of the soil deposit under earthquake conditions. These studies require the development of an appropriate mathematical model of the site, identification of site characteristics expected to influence the response, and estimates of the variability or uncertainty in the particular model parameters related to those characteristics. Considerations in the selection of appropriate mathematical models and subsurface input motions for a site are described in Section 4.3.

The seismologic, geologic, and local soils investigations carried out under Steps 1 and 2 provide a basis for establishing those site characteristics that may have a particular influence on the soil profile response. Uncertainties in such seismologic and geologic site characteristics as the Richter magnitude and epicentral MM intensity of specific earthquake events, and the locations of potential causative faults and tectonic provinces may be considered in the sensitivity studies through appropriate scaling of the subsurface input motions. In addition, those soil profile characteristics whose influence may require evaluation in the sensitivity studies include the depth to rock at the site, the dynamic properties of the various layers within the site, and the orientation, location, thickness, and continuity of the various soil layers. For deep sites with no clear transition from soil to rock, the effective depth of the site may be significant.* Finally, the strength, frequency content, and duration of input motions for the sensitivity studies are particularly important; and, for many sites, variations in these characteristics can have a considerable effect on the resulting soil response analyses. Note that the strength and duration of the input motions should be scaled using the empirical data defined in Step 1, whereas the frequency content of the input motions should be defined from an appropriate ensemble of rock outcrop accelerograms, artificial earthquake records, or computed subsurface rock motions.

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*The effective depth of a deep site is considered to be the depth to rock-like materials. As noted in Chapter 4, these materials are generally considered to have shear-wave velocities exceeding about 2500 fps.

The determination of an appropriate mathematical model is of prime importance in these sensitivity studies. This model must represent the strain dependence of the dynamic stress/strain properties and hysteretic damping characteristics of the soil materials. Furthermore, as discussed in Chapter 4, the model must incorporate those types of seismic waves judged to have the greatest influence on potential earthquake motions at the site. One-dimensional models are most applicable when the site soil deposit is of large lateral extent, has nearly horizontal layering, nearly flat-lying bedrock, and no abrupt changes in layer thicknesses or surface topography over the site. If these conditions are not satisfied, two-dimensional models must be used in the sensitivity studies.

Following a definition of the site model, a series of calculations should be carried out to establish the site-response characteristics as a function of each parameter considered in the sensitivity studies. Results of these analyses may be obtained at a number of locations within the soil deposit and, for each location, may be provided in the form of response spectra, peak accelerations, Fourier spectra, amplification spectra, ground motion time histories, and soil stresses and strains at any location. Statistical measures of site response characteristics (mean, standard deviation, etc.) should be obtained from the results of the various sensitivity studies, in order to provide a quantitative basis for assessing the site-response characteristics.

5.1.4 STEP 4: EVALUATION OF LOCAL SITE RESPONSE CHARACTERISTICS

In this step, the results of the sensitivity studies conducted in Step 3 are evaluated to determine the basic dynamic characteristics of the site and the relative effects of various site parameters on these dynamic characteristics. In addition, an ensemble of response spectra and peak accelerations at appropriate locations within the site profile must be obtained from results that correspond to variations in those site parameters having the greatest influence on the computed soil response. Composite results from this ensemble, in the form of mean and mean-plus-one standard deviation (MSD)* response spectra and peak accelerations, should then be

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*Hereafter in this chapter, spectra or peak accelerations representing the mean-plus-one standard deviation statistical level will be referred to as MSD quantities.

determined. The MSD response spectra are used in Step 6 as a partial basis for defining the criteria spectra for the site, whereas the MSD peak accelerations are used with empirically obtained results from Step 1 to define the criteria peak acceleration for the site.* These peak accelerations are used to scale the site-matched MSD spectra in Step 5 and the Regulatory Guide 1.60 spectra in Step 6.

5.1.5 STEP 5: SELECTION OF SITE-MATCHED RECORDS

Along with the results of site-dependent computations, strong motion measurements from actual earthquakes may also provide an important basis for determination of seismic input criteria for the site. Therefore, in this step, available earthquake ground motion records should be reviewed, and those particular measurements obtained from sites whose seismologic, geologic, and local soil conditions are similar to those for the candidate site should be collected to form an ensemble of site-matched records. Among the site characteristics that should be matched by the selected records are the following:

- a. The region surrounding the accelerograph station at which the records were obtained should have geologic and subsurface soil characteristics that are similar to those of the site being investigated.
- b. The source characteristics for the site-matched records (i.e., capable fault slip mechanism, epicentral distance, and focal depth) should be consistent with those hypothesized for the Safe Shutdown Earthquake for the site.
- c. The Richter magnitude or epicentral MM intensity, as well as the general strength level of the records, should be representative of those parameters estimated for the site.

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*As discussed subsequently in Step 6 (Sec. 5.1.6), site-dependent results are compared to Regulatory Guide 1.60 spectra to define criteria spectra for the site. Therefore, since the Regulatory Guide 1.60 spectra are based on an MSD statistical level from existing earthquake records, a similar statistical level of site-dependent results should be utilized for purposes of this comparison.

Matching as many of the above characteristics as possible will help to assure that the strength, frequency content, and duration of strong shaking of the site-matched records will be representative of ground motions induced by the Safe Shutdown Earthquake for the site. However, as discussed in Chapters 2 and 4, the present limitations in the size of the available library of strong motion records requires that some of these site-matching requirements must be relaxed. Therefore, it is necessary to first define those particular site characteristics judged to have the greatest influence on the ground shaking at the site, and then to select an ensemble of records that best represents these characteristics.

Once an appropriate ensemble of site-matched records has been selected, each member of the ensemble should be scaled so that its peak acceleration corresponds to that defined as the criterion for the site in Step 4. Response spectra corresponding to the mean and mean-plus-one standard deviation (MSD) statistical levels for these scaled records should then be obtained for subsequent evaluation in Step 6.

5.1.6 STEP 6: APPRAISAL AND SELECTION OF SEISMIC INPUT CRITERIA

The final step in this general procedure consists of an appraisal of the site response characteristics as determined from analytical techniques (Step 4) and from site-matched records (Step 5). These response characteristics must be compared to those represented by the site-independent spectrum shapes contained in Regulatory Guide 1.60 to provide a basis for final selection of the seismic input criteria for the nuclear plant site. In this comparison, the Regulatory Guide 1.60 spectra are scaled so that their zero-period acceleration corresponds to the criteria peak acceleration determined for the site in Step 4.

It is apparent that no firm set of rules for carrying out the above comparisons or for defining seismic input criteria can be provided for all sites. For some cases, certain site conditions may necessitate the emphasis of calculations and site-matched records in the criteria selection process, possibly in combination with Regulatory Guide 1.60 requirements. For other sites, it may be shown that the site-independent Regulatory Guide 1.60

spectrum shapes by themselves will be adequate for use as seismic input criteria. Clearly, the final selection of seismic input criteria for a particular site must depend on the characteristics of that site; an assessment of results from site-matched records, analyses, and Regulatory Guide 1.60; and on judgment regarding the merits and limitations of each of these methods when applied to the site. A summary of some of the general merits and limitations of each of the methods is provided in Table 4-3 of Chapter 4.

Although it is not possible to present well-defined rules for selecting seismic input criteria at any given site, it *is* possible and necessary to indicate some of the judgment factors that should be considered in the selection process. Table 5-1 summarizes some of these considerations involved in assessing the applicability of site-response analyses, site-matched records, and the Regulatory Guide 1.60 for any particular site. This table indicates that the selection process depends on such factors as: (1) the availability of site-matched records that represent the most significant seismologic, geologic, and local soil conditions at the site, (2) an assessment of the ability of the soil response analysis to represent the predominant seismic waves at the site, and (3) an evaluation of the applicability of statistically obtained spectrum shapes based on current strong-motion data, but without regard to potential effects from different geologic or local soil conditions.

A final consideration in the process for selecting earthquake input criteria is the form in which the criteria are to be provided. This, in turn, depends on the type of structure analysis that is to be carried out and the technique used to represent soil/structure interaction in the analysis. The following choices are most typically considered:

- a. The most commonly used approach in nuclear plant seismic analyses is to provide earthquake input in the form of appropriately damped response spectra that represent horizontal and vertical earthquake motions. These two spectra may have different damping ratios but are usually applied to soil/structure interaction models in which the soil is represented as a series

TABLE 5-1. FACTORS TO CONSIDER IN FINAL ASSESSMENT OF SEISMIC INPUT CRITERIA

Results	Basic Questions	Considerations in Seismic Input Criteria Assessment
Site-Response Analyses	Adequacy of mathematical model	An assessment of the applicability of analytical results to a given site should consider the kinds of seismic waves that might exert a predominant influence on the ground shaking vs. the type of waves that can be included in the mathematical model. The extent to which the model represents the waves judged to be significant for the site should influence the manner in which the calculations are weighed in the input criteria selection process.
	Sensitivity of results to soil properties and input motions	When calculating results from the site response analyses, the engineer must determine the relative sensitivity of these results to reasonable variations in soil property parameters and subsurface input motions. This should serve as a guide in (1) planning additional calculations that serve to further investigate the sensitivity of the ground motions to any particular important parameters and (2) defining the ensemble of analysis results to be used in the final selection of the input criteria for the site (Step 6).
Site-Matched Records	Seismologic, geologic, and local soil conditions of site-matched records correlated with conditions at site	There are a number of parameters related to the source mechanism, source-site transmission path, and local soil properties that affect the ground shaking at a site. Invariably, it will not be possible to select site-matched records that match all of these parameters; instead, records must be selected that match those particular factors judged most important at the site under investigation. The extent to which each of these parameters for the site-matched records correlate or fail to correlate with those for the nuclear plant site should dictate how these records are weighed in the criteria selection process.
	Availability of suitable records	For certain siting conditions, adequate site-matched records may be scarce. The size of the ensemble of site-matched records should be sufficient to provide a meaningful statistical evaluation of the site response characteristics.
Regulatory Guide 1.60 Spectra	Seismologic, geologic, and local soil conditions at site correlated with norm considered in Regulatory Guide 1.60	The Regulatory Guide 1.60 spectrum shapes, although possibly adequate for a wide range of siting conditions, may not reflect the effects of unusual conditions on earthquake motions at a site. Such unusual conditions that might not be adequately reflected in Regulatory Guide 1.60 include sites that are usually soft, sites with interspersed hard and soft layers, very deep sites, or sites very close to a potential causative fault. For these or other such site conditions, the Regulatory Guide 1.60 spectra may not be representative of ground shaking over important frequency ranges. Therefore, the engineer must judge to what extent his particular site has conditions that may require site-dependent considerations when weighing the potential use of the Regulatory Guide 1.60 spectra in the input criteria selection.

of springs and dashpots at one or more locations along the embedded surface of the structure. For this case, possible variations of earthquake motions with depth are often neglected; therefore, the seismic input is provided as a single family of damped response spectra.

- b. A second approach to nuclear plant structure analysis is to apply horizontal and vertical earthquake motion time histories (instead of response spectra) to the soil springs and dashpots along the embedded portion of the structure. As in Approach *a*, variations of earthquake motions with depth are neglected; therefore, a single horizontal and vertical motion history is utilized for all spring-dashpot locations. Such time histories are typically developed by systematically modifying a selected motion time-history record so that its spectrum is compatible with a criteria spectrum for the site. One method for carrying this out has been developed by Tsai, who has shown that suppression of a portion of the spectrum of a given time-history record can be achieved by passing the selected record through frequency-suppressing filters, while the appropriate segments of the spectrum can be raised by superposing sinusoidal motions (Ref. 5-4).
- c. A third approach for representing seismic input considers the variation with depth of the earthquake ground motions. Input of this type can be applied either to spring-dashpot or finite element models of the soil/structure system. For soil/structure interaction represented by springs and dashpots, the variation with depth of the ground motions must only be considered for a depth below the ground surface equal to the embedded depth of the structure. Where finite element soil/structure models are utilized, input motion time histories are often applied along the entire depth of the soil profile. For either of these

analysis techniques, two methods for specifying input motion time histories may be used:

1. If the input criteria for the site are strongly dependent on results from the soil-response computations, then the depth-dependent motion time histories obtained from these computations can be used as input to the structure response analysis.
2. If the input criteria for the site are more strongly dependent on site-matched records or Regulatory Guide 1.60 spectra (which provide input only at the ground surface), then corresponding time histories below the ground surface must be obtained by computational techniques. Methods for developing subsurface soil motions that are consistent with a predefined surface motion record and the soil conditions at the site have been developed by Schnabel, et al. (Ref. 5-5) and are discussed in Section 4.3.1 of Chapter 4.

5.2 EXAMPLE APPLICATION OF PROCEDURE

In this section, seismic input criteria spectra for an example site are developed in order to illustrate the application of the procedure described in Section 5.1. This particular example is limited to consideration of horizontal ground motion spectra, although the procedure can be extended to include vertical input criteria as well.

The particular site selected for this example consists of deep sand deposits whose densities and stiffnesses exhibit a gradual increase with depth. The Regulatory Guide 1.60 spectra may not be representative of the ground shaking over the entire frequency range of interest for deep cohesionless soil sites of this type (Chap. 4); therefore, this example serves to illustrate the application of the six-step procedure for this situation. As discussed subsequently, the Safe Shutdown Earthquake for the site corresponds to a Magnitude 6.5 event centered 8 mi from the site.

It is noted that the above conditions are analogous to those under which strong motion records were measured at 8244 Orion Blvd. (Los Angeles) during the 1971 San Fernando earthquake. Therefore, the recorded ground motions obtained at the Orion site will serve as a check of the criteria spectra obtained using the six-step procedure.

5.2.1 STEP 1: GEOLOGIC AND SEISMOLOGIC STUDIES

The site considered in this example is located in Southern California, and consists of alluvial deposits that overlie Pleistocene terrace deposits and sandstones. This overall region is underlaid by Tertiary sandstones and shales and ancient dioritic and granitic rock formations.

The region is seismically active and contains a number of strike-slip and reverse faults that have affected the more recent rock formations in the area. Seismological and geological investigations of the region have indicated that the Safe Shutdown Earthquake for the site corresponds to a Magnitude 6.5 centered along reverse faulting that extends toward the site. The center of energy release along this fault plane is considered to be located about 8 mi from the site. The corresponding modified Mercalli intensity at the site is estimated to be VII-VIII.

As part of this step, it is necessary to utilize empirical data to estimate the strength of rock outcrop motions that serve as a basis for defining subsurface input motions for the soil response analyses (Step 3). In addition, empirical data must also be used at this stage to provide estimated values of the duration of strong shaking at the site, as well as first estimates of the peak ground acceleration. As noted in Section 5.1, these peak acceleration estimates will be used in conjunction with results from soil response analyses (Step 4) to provide a final definition of the criteria peak acceleration for the site.

The empirical curves derived by Schnabel and Seed and described in Section 2.3.3 of Chapter 2 are used to scale the strength of the rock motions at the site (Ref. 5-6). These curves indicate that rock outcrop motions 8 mi from the center of a Magnitude 6.5 earthquake should have peak accelerations of about 0.35g (Fig. 5-1a).

First estimates of the peak ground acceleration for the site can be obtained using the empirical data developed by Coulter, et al., (Ref. 5-7) and by Housner (Ref. 5-8). As indicated in Figure 2-2, Coulter, et al., developed intensity-acceleration data for a variety of soil conditions. For this example site, the particular data corresponding to "average foundation conditions" was utilized, and indicated a peak acceleration of about 0.25g for a site intensity of VII-VIII. Housner developed a series of empirical curves that interrelate acceleration, magnitude, and distance. For a site located 8 mi from the center of a Magnitude 6.5 earthquake, Housner's data indicates a peak acceleration of about 0.3g. Therefore, from appropriate empirical data, first estimates of peak acceleration of 0.25 to 0.3g are indicated.

For estimating the duration of strong shaking, the data of either Housner (Ref. 5-8) or Bolt (Ref. 5-9) can be used. Both sets of data indicate a 15- to 20-sec duration of strong shaking at the site (Fig. 5-2).

5.2.2 STEP 2: SITE SOILS INVESTIGATIONS

The soil profile for the site consists of deposits of dense and gravelly sands that extend to sandstone and shale rock at a depth of about 800 ft (Fig. 5-3). These deposits exhibit shear wave velocities that increase with depth to maximum values in the neighborhood of 2000 ft/sec at their interface with the rock 800 ft below the ground surface. The rock beneath the soil deposits has a shear wave velocity of 2800 ft/sec. The layering characteristics and boundaries in the vicinity of the site can be considered reasonably horizontal and uniform in nature.

FIGURE 5-1. EMPIRICAL SCALING CURVES FOR DEFINING PEAK ACCELERATIONS AT EXAMPLE SITE

(a) HOUSNER RESULTS
(REF. 5-8)

(b) BOLT RESULTS (REF. 5-9)

FIGURE 5-2. EMPIRICAL SCALING CURVES FOR DEFINING DURATION OF STRONG SHAKING AT EXAMPLE SITE

NOTE:

 γ = UNIT WEIGHT K_0 = COEFFICIENT OF EARTH
PRESSURE AT REST

(a) SOIL MATERIALS

(b) SHEAR WAVE VELOCITIES

(c) SHEAR MODULUS AT SMALL STRAINS (ρV_s^2)

FIGURE 5-3. SOIL PROFILE AND MATERIAL PROPERTIES FOR EXAMPLE SITE

Strain-dependent dynamic shear moduli and damping factors for the various soil materials must also be estimated for use in the soil response analyses. For purposes of this example, these strain-dependent properties are defined by the Seed-Idriss empirical data shown in Figure 5-4 (Ref. 5-3).*

It is judged that the primary uncertainties in the site soil properties described above are associated with the stiffness of the dense-to-gravelly sands extending from 250 to 800 ft below the ground surface. To account for these uncertainties, two sets of soil properties will be considered in the site response sensitivity studies (Step 3). The first, termed the "lower-bound soil properties," utilizes those properties, defined in Figure 5-3 and described above. The second set of properties, termed "upper-bound soil properties," considers shear moduli for the dense-to-gravelly sands to be increased by 50%. All other properties for site soil materials are assumed to remain unchanged.

5.2.3 STEP 3: SITE RESPONSE SENSITIVITY STUDIES

As indicated in Step 2 above, the layering characteristics and boundaries in the vicinity of the site are reasonably horizontal and uniform in nature. Therefore, it is judged appropriate to carry out sensitivity studies of the horizontal response of the site using a one-dimensional model based on vertically propagating shear waves that account for the strain-dependence of the soil material properties. A number of appropriate computer programs satisfy these requirements, although for this particular example, the SHAKE program developed at the University of California at Berkeley (Ref. 5-10) is utilized.

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*At an actual nuclear plant site, the strain-dependent soil properties would be established from laboratory tests of the soil materials (see Chap. 3).

(a) STRAIN-DEPENDENT DAMPING RATIO

(b) STRAIN-DEPENDENT SHEAR MODULUS

NOTE

1 THESE SHEAR MODULI AND DAMPING CURVES ARE FOR SAND MATERIALS. IF MATERIALS ALSO CONTAIN CLAYS AND/OR GRAVELS, IN ADDITION TO THE SANDS, THE CURVES MAY REQUIRE APPROPRIATE SCALING.

2 FOR A GIVEN LAYER, THE SHEAR MODULUS, G , CAN BE OBTAINED FROM THE RELATIVE DENSITY, D_r , OR ALTERNATIVELY, FROM THE FOLLOWING PROCEDURE:

- a. BASED ON THE MASS DENSITY, ρ , AND THE SHEAR WAVE VELOCITY, V_s , COMPUTE THE SHEAR MODULUS, G_0 , AT SMALL STRAINS (10^{-4} PERCENT) AS

$$G_0 = \rho V_s^2$$

- b. BASED ON THE POSITION OF THE LAYER WITHIN THE SOIL PROFILE, AND THE DENSITY OF THE OVERLYING MATERIALS, THE MEAN EFFECTIVE STRESS, σ'_m , CAN BE DETERMINED FOR THIS GIVEN LAYER.

- c. THE PARAMETER K_2 AT SMALL STRAINS CAN THEN BE DETERMINED FROM G_0 AND σ'_m AS

$$K_2 = \frac{G_0}{1000 (\sigma'_m)^2}$$

- d. STEPS a THROUGH c, RATHER THAN THE RELATIVE DENSITY APPROACH, HAVE BEEN USED TO SELECT THE APPROPRIATE STRAIN-DEPENDENT SHEAR MODULUS CURVE FOR EACH LAYER IN THE SITE 1 ANALYSES.

FIGURE 5-4. DATA FOR DETERMINATION OF STRAIN-DEPENDENT SOIL PROPERTIES FOR EXAMPLE SITE ANALYSIS (REF. 5-3).

The analytical approach used to compute ground surface responses is illustrated in Figure 5-5 and consists of the following four steps:

Step A -- Original Rock-Outcrop Motions: Select a suitable ensemble of rock-outcrop motions for use at the example site.

Step B -- Scaled Rock Outcrop Motions: Using the attenuation data shown in Figure 5-1a, scale the amplitudes of each member of the ensemble to correspond to rock-outcrop motions at a distance of 8 mi from the center of energy release of a Magnitude 6.5 earthquake.

Step C -- Subsurface Rock Motions: Apply deconvolution techniques described in Reference 5-5 to compute an ensemble of subsurface rock motions from the ensembles of scaled rock-outcrop motions determined in Step B. For this example site, the subsurface motions are assumed to act at a depth of 800 ft, along the interface of the dense-to-gravelly sand and the soft rock material. This same soft rock material is assumed to comprise the rock outcrop through which the deconvolution process occurs.

Step D -- Ground Surface Motions: With the above ensemble of subsurface rock motions as input, compute the response of a one-dimensional continuum model of the site profile. This site model utilizes strain-dependent shear moduli and damping ratios for the various soil materials, according to the empirical relationships shown in Figure 5-4. As shown in Figure 5-5, the soft rock below the point of application of the subsurface input motions is also included in the model in order to represent the effects of radiation damping and the deformability of this underlying rock material.

FIGURE 5-5. ANALYTICAL TECHNIQUE USED TO COMPUTE GROUND SURFACE RESPONSE AT EXAMPLE SITE

The primary variables to be considered in the sensitivity studies for this example site are the rock-outcrop motions and the subsurface soil properties. Variations in the subsurface soil properties are represented by utilization of the upper-bound and lower-bound properties indicated in Step 2 (Sec. 5.2.2). The ensemble of rock-outcrop motions used for these studies are comprised of the two horizontal components of motion recorded at Pacoima Dam and Castaic during the 1971 San Fernando earthquake. These accelerograms were each recorded on rock or extremely stiff site conditions and correspond to reverse fault slippage as the source mechanism. In addition, the duration of strong shaking of these records is within the range indicated in Figure 5-2. Each record was scaled to a peak acceleration of 0.35g, in order to represent rock motions 8 mi from the center of a Magnitude 6.5 earthquake (Fig. 5-1a)*. Response spectra for this ensemble of scaled records are shown in Figure 5-6.†

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* It is noted that the peak accelerations from both horizontal components of motion recorded at Pacoima Dam exceeded 1g. However, these peak accelerations correspond to high frequency spikes that are not representative of the strength of these ground motions in the range of frequencies important to nuclear plant structures. Therefore, the peak acceleration of each Pacoima Dam record was truncated to a value of 0.9g, and a scale factor of $0.35/0.90 = 0.39$ was applied to represent an effective peak acceleration of 0.35g for these records. The peak accelerations for both Castaic records were considered directly in the scaling process without any truncation.

† Under action of the Safe Shutdown Earthquake, USNRC Regulatory Guide 1.61 indicates damping ratios ranging from 0.02 to 0.07 for the various components of nuclear plants (Ref. 5-11). Therefore, for this example, response spectra corresponding to a damping ratio of 0.05 are provided, since this representative value falls within the above indicated range. For an actual nuclear plant, however, seismic input criteria spectra should be provided for a range of damping ratios extending from 0.005 to as high as 0.10.

FIGURE 5-6. SPECTRA OF ROCK-OUTCROP MOTIONS USED AS INPUT TO SITE RESPONSE ANALYSES

The effects of variations in the rock outcrop motions on the computed ground response spectra are shown in Figure 5-7, and the sensitivity of the ground response spectra to variations in the soil properties is shown in Figure 5-8. These figures indicate the following trends:

- a. The ground surface response spectra are sensitive to the frequency content of the rock-outcrop motions (Fig. 5-7).
- b. The ground response spectra, when compared to the spectra from the rock-outcrop motions, have increased lower-frequency components of motion and attenuated higher-frequency components (Fig. 5-8).
- c. The use of the upper-bound vs. lower-bound soil properties does not have a strong influence on the overall characteristics of the ground response spectra (Fig. 5-8).

The effect of variations in the rock-outcrop motions and soil properties on the peak ground accelerations is shown in Table 5-2. This table indicates that the upper-bound soil properties result in peak accelerations ranging from 8% to 25% higher than those obtained using the lower-bound properties. In addition, for a given set of soil properties, the Castaic input motions result in slightly higher peak ground accelerations than does the Pacoima Dam input. Both Castaic records result in about the same computed peak ground acceleration, whereas the S74W component at Pacoima Dam provides larger peak accelerations than the S16E component. For all cases, the computed peak ground accelerations are less than the 0.35g acceleration level of the rock outcrop input motions.

To obtain composite peak acceleration results for the site, the ensemble of peak accelerations from each computation were analyzed statistically. Results from this analysis indicate a mean value of peak ground acceleration of about 0.25g and an MSD value of 0.28g (Table 5-2). These composite results will be used in Step 4, along with the first estimates of peak acceleration from Step 1, to define a criteria peak acceleration for the site.

(a) UPPER-BOUND SOIL PROPERTIES

(b) LOWER-BOUND SOIL PROPERTIES

FIGURE 5-7. EFFECT OF ROCK-OUTCROP MOTIONS ON COMPUTED GROUND RESPONSE SPECTRA

(a) ROCK-OUTCROP MOTIONS = PACOIMA DAM S16E

(c) ROCK-OUTCROP MOTIONS = CASTAIC N21E

(b) ROCK-OUTCROP MOTIONS = PACOIMA DAM S74W

(d) ROCK-OUTCROP MOTIONS = CASTAIC N69W

FIGURE 5-8. EFFECT OF SOIL PROPERTIES ON COMPUTED GROUND RESPONSE SPECTRA

TABLE 5-2. EFFECT OF ROCK-OUTCROP MOTIONS AND SOIL PROPERTIES
ON PEAK GROUND ACCELERATIONS

Soil Properties	Rock-Outcrop Motions		Peak Ground Acceleration, g
	Record	Component	
Upper Bound	Pacoima Dam	S16E	0.240
		S74W	0.269
	Castaic	N21E	0.284
		N69W	0.283
Lower Bound	Pacoima Dam	S16E	0.196
		S74W	0.249
	Castaic	N21E	0.250
		N69W	0.243

NOTE: Composite results from ensemble of computations are as follows:

Mean = 0.252g

MSD = 0.280g

5.2.4 STEP 4: EVALUATION OF LOCAL SITE RESPONSE CHARACTERISTICS

The sensitivity studies carried out in Step 3 investigated the effects of variations in rock-outcrop motions and soil properties on the computed ground response spectra and peak accelerations. The ground response spectra were shown to be dependent on the frequency content of the rock outcrop motions and less dependent on whether the upper-bound or lower-bound soil properties were used in the site response analyses. The peak ground accelerations were moderately influenced by the particular set of soil properties used in the site model and by the rock-outcrop motions.

An important trend shown by these sensitivity studies was the rather consistent amplification characteristics of the local soil materials. When compared to the spectra of rock-outcrop motions, the computed ground response spectra exhibit amplified low-frequency signals (for frequencies less than about 1 Hz) and reduced higher-frequency motions and peak accelerations. As discussed in Section 4.2, this is consistent with observed trends regarding comparisons of strong earthquake motions recorded on rock and deep soil sites.

Based on the above results, it is necessary during this step to provide a final definition of the peak ground acceleration for the site. Empirical data provided in Step 1 has indicated peak accelerations for the site that range from 0.25g to 0.30g. Although these data are appropriate for providing general guidelines, it is judged that peak accelerations obtained from the site-response analysis represent the more reliable estimates for this example site.* In particular, a peak acceleration of 0.28g, which corresponds to the ensemble MSD from the site response analysis, is selected as the criterion for the site. This MSD acceleration level is consistent with the use of MSD spectra in subsequent comparisons with the Regulatory Guide 1.60 spectra in Step 6.

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*See Chapter 2 for a discussion of uncertainties associated with the use of acceleration/intensity or acceleration/magnitude/distance correlations.

During this step, it is also necessary to develop ensemble mean and MSD spectra from the analytical results. Of particular importance is the MSD spectrum that is used to represent the site response analysis results in the subsequent input criteria definition for the site (Step 6). The ensemble of spectra used for this purpose corresponds to the eight spectra shown in Figures 5-7 and 5-8. These spectra correspond to two different site models (represented by the upper-bound and lower-bound soil properties), each analyzed for four different rock-outcrop motions. The resulting mean and MSD spectra from this ensemble are shown in Figure 5-9. It is noted that the ensemble mean spectrum has a zero-period acceleration of 0.25g (corresponding to the mean peak acceleration indicated in Table 5-2), whereas the ensemble MSD spectrum has a zero-period acceleration of 0.28g that corresponds to the criteria peak acceleration for the site.

5.2.5 STEP 5: SELECTION OF SITE-MATCHED RECORDS

As discussed in Section 4.2 of Chapter 4, the ensemble of site-matched records should be selected to represent as many of the features of the site and the SSE magnitude, distance, and source mechanism as possible. For the site considered in this particular example, it was not possible to define records to represent all of these features. However, effects of local soil conditions were considered to be of particular importance at this example site, and an ensemble of 30 records used by Seed, et al., to depict the response of deep cohesionless soil sites was judged to be generally representative of these soil conditions (Ref. 5-12). Therefore, this ensemble, scaled to be consistent with the peak acceleration criterion defined in Step 4, was selected as the set of site-matched records for the site. Ensemble mean and MSD spectra from these records are provided in Figure 5-10. The scaled ensemble MSD spectrum will be used to represent the site-matched records in the selection of the site input criteria in Step 6.

FIGURE 5-9. ENSEMBLE MEAN AND MSD SPECTRA FROM SITE RESPONSE ANALYSES

FIGURE 5-10. ENSEMBLE MEAN AND MSD SPECTRA FROM SITE-MATCHED RECORDS

5.2.6 STEP 6: APPRAISAL AND SELECTION OF SEISMIC INPUT CRITERIA

To complete the final selection of the seismic input criteria, it is first necessary to compare the ensemble spectra from the site-matched records and the site-response analyses. This comparison of ensemble mean and MSD spectra (Fig. 5-11) indicates a generally good correlation between the analysis results and the site-matched records. For frequencies higher than about 4 Hz, the site-matched spectra tend to exceed the spectra from the analysis results, whereas at lower frequencies, the spectra from the two sets of results compare very closely.

These site-dependent results must next be compared with the spectrum from Regulatory Guide 1.60 scaled to a zero-period acceleration of 0.28g. This comparison (Fig. 5-12) indicates that the Regulatory Guide 1.60 spectrum exceeds the site-dependent spectra at frequencies above about 8 Hz and falls below the site-dependent spectra at lower frequencies. In the frequency range of primary interest for nuclear plant design (2 to 10 Hz), differences between the criteria spectrum and Regulatory Guide 1.60 are slight, but still noticeable.

Causes for the above differences can be attributed to the nature of the site conditions for the records used to develop Regulatory Guide 1.60. As shown in Table 4-1 of Chapter 4, approximately one-third of these records were measured at deep-cohesionless soil sites; the remaining records were measured either on rock or stiff-soil sites whose depths to rock are considerably less than at the example site. Therefore, the Regulatory Guide 1.60 spectrum shapes are biased toward stiffer site conditions that, when compared to spectra representing deep sites, should tend to show amplified higher-frequency components of motion and reduced lower-frequency components; these general trends are evident in Figure 5-12a. Furthermore, the close agreement between site-dependent results from two independent sources, site-matched records and analytical results, suggests that a reasonable degree of confidence can be placed in these spectra when developing seismic input criteria. Consequently, the site-dependent results are

(a) MEAN SPECTRA

(b) MSD SPECTRA

FIGURE 5-11. COMPARISON OF SPECTRA FROM SITE-MATCHED RECORDS AND SITE-RESPONSE ANALYSES

(a) COMPARISONS OF SPECTRA FROM SITE-DEPENDENT RESULTS AND REGULATORY GUIDE 1.60

(b) DEVELOPMENT OF CRITERIA SPECTRUM

FIGURE 5-12. CRITERIA SPECTRUM FOR EXAMPLE SITE

(c) COMPARISON OF CRITERIA SPECTRUM WITH REGULATORY GUIDE 1.60

FIGURE 5-12. (CONCLUDED)

judged to be most representative of the earthquake-induced ground shaking for the example site, and the criteria spectrum for the site is selected to be a smoothed best fit of the site-dependent spectra (Fig. 5-12b). It is noted that this criteria spectrum exceeds the Regulatory Guide 1.60 spectrum in the lower-frequency range and falls below the Regulatory Guide 1.60 spectrum at higher frequencies (Fig. 5-12c).*

As indicated previously, the conditions under which the Orion records were obtained (during the 1971 San Fernando earthquake) are analogous to those of the example site; therefore, the Orion spectra can be used as a check on the seismic input criteria obtained in this example. As shown in Figure 5-13a, the mean spectra from the site-dependent results compare reasonably well with the Orion spectra. The criteria spectrum from the site, which represents an MSD statistical level from the site-dependent results, is seen to be conservative when compared to the Orion spectra, but not unreasonably so (Fig. 5-13b). The Regulatory Guide 1.60 spectrum is seen to exceed the Orion spectra at higher frequencies, and falls below certain regions of the Orion spectra in the low- to mid-frequency range (Fig. 5-13c).

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*In Regulatory Guide 1.60, it is recognized that the spectra presented in that document "may not apply to sites that (1) are relatively close to the epicenter of an expected earthquake or (2) that have physical characteristics that could significantly affect the spectral combination of input motions." For either of these conditions, Regulatory Guide 1.60 requires that "the Design Response Spectra...should be developed on a case-by-case basis." Clearly, the site selected for this particular example has physical characteristics, such as the presence of very deep soil deposits, which affect the resulting ground surface response spectra. As a result, this example represents an exception to the general site conditions specified in the Regulatory Guide for use of its standardized spectrum shapes, and is in line with the requirements of Regulatory Guide 1.60 for these exceptions.

(a) MEAN SITE-DEPENDENT SPECTRA

(b) CRITERIA SPECTRUM

FIGURE 5-13. SPECTRUM COMPARISONS: RESULTS FROM EXAMPLE SITE VERSUS ORION RECORDS

(c) REGULATORY GUIDE 1.60

FIGURE 5-13. (CONCLUDED)

5.2.7 SUMMARY OF EXAMPLE

In summary, the procedure described in Section 5.1 has been used in this example to develop seismic input criteria for a deep-soil site and an SSE event that is analogous to conditions under which the Orion records were obtained during the 1971 San Fernando earthquake. An application of results from site-response analyses, site-matched records, and the Regulatory Guide 1.60 has provided a seismic input criteria spectrum that is conservative, but not unreasonably so, when compared to the Orion spectra. The Regulatory Guide 1.60 spectrum was judged to be less applicable to a deep cohesionless soil site of this type than were site-dependent spectra from analyses and site-matched records.

It should be noted that these results and trends are based solely on this particular example and should not necessarily be considered applicable to other sites or local soil conditions. For example, although the Regulatory Guide 1.60 spectra were judged to be less applicable than the site-dependent results for representing ground motions at this example site, they may be more nearly representative of ground shaking at nuclear plant sites with stiffer and shallower soil deposits. Also, the particular manner in which the results from site-matched records, site response analyses and Regulatory Guide 1.60 were weighed to develop the final input criteria for this example site should not be considered universal. The procedure described and illustrated in this chapter provides a rational basis for defining seismic input only if careful judgment is applied on a site-by-site basis.

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