SEISMIC DESIGN TECHNOLOGY FOR BREEDER REACTOR STRUCTURES

Volume 1: Special Topics in Earthquake Ground Motion

AGBABIAN ASSOCIATES
El Segundo, CA 90245

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Volume 1: Special Topics in Earthquake Ground Motion

April 1983

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PREFACE

In 1974 Agbabian Associates embarked on a project to develop and update seismic design criteria and analysis methods as part of the national long-range program on Breeder Reactor Seismic Technology. This work was initiated by the Division of Reactor Development and Demonstration of the Energy Research and Development Administration. The work was continued under contract with the Department of Energy (DOE) and under contract with Union Carbide Corporation. The Technical Monitors for the project were C. Chester Bigelow for DOE and Claude Pugh and Richard Gwaltney for Union Carbide. The Project Managers at Agbabian Associates were Dr. George A. Young and Dr. Damoder P. Reddy.

The purpose of the study was to identify the problem areas associated with the application of seismic hazard assessment procedures and to establish guidelines for design and analysis of earthquake-resistant breeder reactor structures, equipment, and piping. The result of these research efforts was a series of thirty topical reports covering various topics related to the overall objective; these reports were produced over the span of years 1975-82.

The present volume is part of a subsequent shorter series that condenses the information from the original volumes into four volumes. There are three reasons for this condensed version. The first is that the original volumes were published on a limited distribution basis, and it is intended that the information published in the present volumes be available for a wide distribution and greater accessibility. Second, the information in the new, shorter form will be easier to use and therefore will find wider reception and greater utility in this way. Third, since much of the information contained in the topical reports has

general application, the text of these four volumes is prepared to serve as a useful reference on many general topics in the analysis and design of earthquake-resistant nuclear facilities.

The general title for the four volumes is Seismic Design Technology for Breeder Reactor Structures, and the volumes are entitled:

- Vol. 1. Special Topics in Earthquake Ground Motion
- Vol. 2. Special Topics in Soil/Structure Interaction
- Vol. 3. Special Topics in Reactor Structures
- Vol. 4. Special Topics in Piping and Equipment

The present volume, Volume 1, consists of twelve chapters which cover these respective topics pertaining to nuclear power plants:

<u>Chapter</u>	<u>Topic</u>
1	Seismic Hazard Analysis Procedures
2	Statistical and Probabilistic Considerations in Hazard Analysis
3	Vertical Ground Motion Characteristics
4	Vertical Ground Response Spectrum Shapes
5	Effects of Inclined Rock Strata on Site Response
6	Correlation of Ground Response Spectra with Intensity
7	Intensity Attenuation Relationships
8	Peak Ground Acceleration in the Very Near Field
9	Statistical Analysis of Response Spectral Amplitudes
10	Contribution of Body and Surface Waves

- 11 Evaluation of Ground Motion
 Characteristics
- 12 Design Earthquake Motions

These chapters are based on twelve topical reports SAN/1011-101, SAN/1011-104, SAN/1011-107, SAN/1011-108, SAN/1011-109, SAN/1011-113R, SAN/1011-114, SAN/1011-119, SAN/1011-124, SAN/1011-125, SAN/1011-126, and DOE/SF/01011-129.

Technical contributions to these twelve

topical reports were made by M.S. Agbabian, Y.C. Lee, D.P. Reddy, S.D. Werner, H.S. Ts'ao, and G.A. Young. The four volumes were produced under the general direction of M.S. Agbabian; technical editor was J.W. Athey. While preparing these volumes, Agbabian Associates received guidance from C. Chester Bigelow and Richard Gwaltney. It is hoped that these volumes will be a useful reference on topics that are essential for the safe design of nuclear facilities.

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CHAPTER 1

SEISMIC HAZARD ANALYSIS PROCEDURES

1.1 INTRODUCTION

Problems associated with the practical application of seismic hazard analysis procedures to nuclear power plant design must be identified, and the need for acceptable procedures to perform such analyses established. The requirement for a statistical assessment of the seismic hazard of a proposed nuclear plant site is not defined in the federal regulations, but is implied by the high standards of safety and reliability required for containment and safe shutdown of the reactor during and following a major seismic event. Lack of a clear understanding of this requirement results from the fact that the procedures set forth in the Code of Federal Regulations, 10 CFR 100, Appendix A (USAEC, 1973a) for the selection of design earthquakes for nuclear power plant sites are completely deterministic and make no provision for the statistical determination of the seismic hazard. However, determining the reliability achieved in the design of the safety related systems requires that the probability of experiencing the design earthquakes during the service life of the facility be determined.

Since deterministic procedures are used to establish design earthquakes, nuclear power plants are currently being designed to different levels of seismic risk. Even though the level of reliability achieved in the design of all plants may be satisfactory, the question that invariably arises is whether the design earthquakes should be selected by probabilistic rather than by deterministic procedures so that all plants can be designed to the same level of seismic risk. It may then sometimes be inferred that the provisions of 10 CFR 100, Appendix A, are incorrect. Actually, there are physical factors

that limit the magnitudes of earthquakes that can be expected to occur in different regions of the world, and of the United States. Therefore, the establishment of the maximum credible earthquake environment expected at a proposed nuclear plant site by the procedures defined in 10 CFR 100, Appendix A, is a necessary and important step in determining the seismic hazard associated with a proposed site. Furthermore, these procedures are not in conflict with the statistical relationships then used to provide a probabilistic assessment of the site. The only step that may require modification is the arbitrary use of the maximum credible earthquake environment as the Safe Shutdown Earthquake (SSE). In contrast, perhaps an acceptable seismic risk level should be specified for the SSE, and the requirements of a seismic hazard assessment procedure should then be clearly defined.

Code of Federal Regulations, 10 CFR 100, Appendix A, differentiates between three different tectonic conditions in defining procedures for establishing the SSE: (1) sites for which the historic earthquakes of greatest magnitude cannot be related to known tectonic structures, but can be identified with the tectonic province in which the site is located, (2) sites for which the historic earthquakes of greatest magnitude cannot be related to known tectonic structures but can be identified with tectonic provinces in which the site is not located, and (3) sites for which the historic earthquakes of greatest magnitude can be correlated with known tectonic structures. The seismic hazard analysis for the first condition can be performed by a uniform seismicity approach, while the two remaining conditions require a variable seismicity approach.

1.2 UNIFORM SEISMICITY APPROACH

Seismic hazard assessment using the uniform seismicity approach was first described by Housner (1969; see also Chapter 2 below). This approach is used for sites in regions where the historic earthquakes of greatest magnitude cannot be related to known tectonic structures but can be identified with the tectonic province of the site. The seismic history of the region must indicate that two important probabilistic assumptions are reasonably well satisfied: that there is an equal probability that a given seismic event can occur anywhere within the seismic region being considered, and that the arrival rates of the earthquake events are independent of each other and are Poisson distributed. By judicious selection of the contributing seismic area, and of the recurrence data, results can usually be obtained that are adequate for design even though these assumptions may not be completely satisfied. The application of the procedure involves the following steps:

- a. Review the geology and seismic history of the region to establish that the uniform seismicity assumptions are reasonably well satisfied.
- b. Develop a relationship between the earthquake recurrence rate and earthquake magnitude (or epicentral intensity).
- c. Predict the maximum earthquake magnitude (or epicentral intensity) potentially possible in the region.
- d. Develop an earthquake intensity attenuation relationship with distance from the epicenter.
- e. Calculate the probability of occurrence of different intensity levels at any point (i.e., nuclear power plant site) within the region investigated during given periods of time.

An example treatment of this procedure using the southern Appalachian seismic zone (Fig. 1-1) defined by Bollinger (1973) is presented here.

FIGURE 1-1. GEOLOGIC PROVINCES IN SOUTHEASTERN UNITED STATES

1.2.1 REGIONAL GEOLOGY

In order to provide a seismic hazard assessment for a specific zone it may be necessary to consider an area larger than the seismic region or geologic province considered when establishing the Safe Shutdown Earthquake (SSE) and Operating Basis Earthquake (OBE). Therefore, the seismic hazard study will start with consideration of a larger area than the southern Appalachian seismic zone.

The region of interest to this study can be divided into five geologic provinces having similar geologic history, lithology, and structures (Rodgers, 1970; Fisher et al., 1970; Fenneman, 1938).

a. Appalachian Plateau: This province is characterized by nearly flat lying sedimentary rocks of Paleozoic age which have been deeply dissected by erosion. There are some gentle folds and a few faults. The province represents the portions of eastern Tennessee and Kentucky that extend from the eastern flank of the Nashville Dome and Cincinnati Arch east to the Pine Mountain Fault.

b. Valley and Ridge: This province is characterized by steep northeast-southwest trending ridges separated by gentle rolling valleys. It is composed of Paleozoic sedimentary rocks that have been extensively uplifted, folded, and faulted as previously described. In general, flood plains of the major streams are narrow, and only limited alluvial soils are present. The province extends from the Pine Mountain and St. Clair Fault lines on the west to the exposure of the Precambrian metamorphic rocks on the east.

c. Blue Ridge: This province consists of metasedimentary and metavolcanic rocks of Precambrian age that were intensely folded and metamorphosed during late Precambrian time. These structures were greatly modified by middle and late Paleozoic tectonic events. This

province is defined on the east in North Carolina by the Brevard fault zone.

d. Piedmont: This province extends from the Brevard fault zone in the west to the base of the Cretaceous sedimentary rocks in the coastal plains area in the east. It is characterized by intensely deformed and metamorphosed Paleozoic rocks.

e. Coastal Plains: The coastal plains province can be defined as that portion of the continental shelf that has emerged from the ocean without substantial deformation. It consists of gently sloping sediments of Cretaceous and Tertiary age.

The geologic history of the Appalachian region can be briefly summarized by starting with the early Cambrian period of the Paleozoic era (600 million years ago). At this time in geologic history, mountain building was occurring to the northeast in New England. For the next 300 million years, there was a nearly unbroken period of marine sedimentation in the Appalachian and coastal region. This is indicated by the deep deposits of limestones, dolomites, sandstones, and shales that have been identified geologically with this era. The Appalachian Orogeny occurred near the end of the Paleozoic era (220 to 270 million years ago). During this 50 million year period, large compressive forces from the southeast crushed and folded the earth's crust along the east coast of the United States, forming the Appalachian Mountains which now extend from Newfoundland in the northeast to as far southwest as central Alabama. Rock units were folded first into major and minor synclines and anticlines. This was followed by low angle thrust faulting which forced older rocks over younger sediments in many areas. During the millions of years that have followed, the mountains have eroded and the older rock units are now exposed. This general condition exists throughout the region.

The Saltville and Pulaski faults are examples of two major northeast-southwest trending faults. The cause of the tectonic forces generating the intense folding and faulting in this area during the Appalachian Orogeny has been attributed (Hatcher, 1972) to the collision of the North American and African Continental plates. The forces responsible for the deformation are considered to be no longer active since the continents have drifted apart. It has been postulated that present earthquake activity in this region results from the release of residual stresses. Because of the many existing fault planes and weak rock layers, it has been postulated that stress is relieved before excessive strain energy accumulates. Hence the resulting earthquakes are generally of small magnitude. It has also been noted (Bollinger, 1973) that present-day earthquakes occur in four of the geologic provinces and the activity is oriented both parallel and transverse to the dominant northeast-trending structures. Also, the many thrust and normal faults that have been mapped in each of the provinces do not have a record of surface breakage during historic time. It is therefore generally held that the seismic activity is independent of the geologic provinces and may not be associated with the exposed geologic structures but is centered below the exposed rock structures. The exact cause and most probable location of future earthquakes in the southeastern United States is, therefore, not known. For example, the largest historical earthquake in southeastern United States, the Charleston, South Carolina earthquake in 1886, is associated with completely buried tectonic structures. Whether an earthquake of this magnitude can occur elsewhere in this general region is a matter of considerable speculation.

1.2.2 SEISMIC HISTORY

A large contributing area or a long period

of records, or both, are required to provide sufficient data for establishing a stable earthquake frequency of recurrence curve. Unfortunately, expanding the area to include more seismic events invariably reduces the homogeneity of the area and invalidates the assumption of equal probability of occurrence of a seismic event anywhere within the area. Considering a small area initially, even though adequate records appear to be available, may also be misleading if the period of records is short and an abnormal period of seismic activity has occurred in the smaller area during the historical period. Hence, the average seismicity for a larger area may in some cases be a better estimate for the future, than the average for the smaller area. This can be demonstrated by the seismic history of South Carolina. For 200 years prior to the Charleston earthquake of 1886, there had been little seismic activity in the Charleston area. Following this earthquake, over 300 separate seismic events were experienced in a period of 30 years. During this period, the central and northwestern part of South Carolina was seismically relatively inactive. More recently, the seismic activity has increased in these areas and the Charleston activity has diminished (Bollinger, 1972).

An additional factor that must be considered is the completeness of the seismic record. While a few major cities on the East Coast were settled before 1700, much of the inland region was relatively unsettled until after 1870, and adequate instrumentation to detect low magnitude earthquakes still does not exist in most areas. Therefore, a careful review must be made of the seismic history prior to the development of a recurrence curve.

1.2.3 EARTHQUAKE FREQUENCY OF RECURRENCE CURVES

In the opinion of leading seismologists (Richter, 1958; Sykes, 1965; Isacks et al.,

1968; Evernden, 1970) the recurrence curve for earthquakes for a given area nearly anywhere in the world can be expressed by an equation of the form

$$\log N = a - bM \quad (1-1)$$

where N is the number of events in a given period of years of local magnitude M , and a and b are constants.* (This equation can also be expressed in terms of epicentral Mercalli Intensity I_0 , or body wave magnitude m_b). Studies have demonstrated that b , which is the slope of the line defining the relationship, will vary between 0.8 and 1.1 when expressed in terms of earthquake magnitude, and when a large number of earthquake events are available for the study. Also, a value of about 0.9, based on world data, is frequently used to constrain the earthquake frequency curve, with the position of the curve on the magnitude axis being constrained by historical earthquakes of Magnitude 4.5 or greater (Couch et al., 1972). This procedure has been followed when adequate instrumentation records of earthquake occurrence and magnitude are not available for a given area. The major reason is that in the absence of instrument records, and with low population density, small magnitude earthquakes frequently go undetected. The recurrence rate of the smaller earthquakes are therefore underestimated. Curves based on such data will have slopes that are too flat. Predictions made from such curves will then overestimate the frequency of recurrence of the larger magnitude earthquakes.

*In this equation, N is also sometimes taken as the cumulative number of events in a given period of years of magnitude M or greater, or as the number of events in a given period of years of local magnitude $M \pm \Delta M/2$ where ΔM is a selected magnitude interval. For all cases, b remains unchanged, but a slightly different constant a results.

Stepp (1972) demonstrated that the period of records and the size of the area are important in the development of the recurrence curve. If the period is short or the area is small, there may not be an adequate sample and unstable predictions will result. Also, it is held by some that since the recurrence curve represents the rate at which different quantities of strain energy are released in the earth's crust, there may be a minimum sized area below which a well defined relationship cannot be developed. Experience indicates that the larger the area the more stable the relationship will be, but the areal distribution of the seismic events will be less homogeneous. Stepp developed procedures for testing the stability of the analysis and for establishing a better and more stable recurrence curve than can be obtained by considering all data with no adjustment. These procedures have been considered by Bollinger in the development of a recurrence curve for the southeastern United States region.

Bollinger tested the stability of the data in Table 1-1 for the last 100-year period using the procedures of Stepp and found that the Intensity V data had been reasonably stable for the entire 100 years, Intensity VI for the past 60 years, and Intensity VII for the past 40 years. Intensity VIII had poor stability for the entire period. Based on Stepp's procedures, the following frequency of recurrence equation was developed for the southeastern United States seismic region by Bollinger:

$$\log N = 3.01 - 0.59 I_0 \quad (1-2)$$

where N = number of events per year and I_0 = epicentral Mercalli Intensity. When I_0 is expressed in terms of Richter Magnitude ($M = 1 + 2/3 I_0$), the following is obtained:

$$\log N = 3.90 - 0.89 M \quad (1-3)$$

These equations are based on a total area of 135,000 sq mi.

TABLE 1-1. NUMBER OF EARTHQUAKES BY DECADE AND BY SIZE FOR THE SOUTHEASTERN UNITED STATES (Bollinger, 1973)

Decade	?	III	IV	V	VI	VII	VIII	IX-X	Row Totals
1770-79	1	-	1	2	-	1	-	-	5
80-89	2	-	-	-	-	-	-	-	2
90-99	6	-	1	-	-	-	-	-	7
1800-09	1	-	-	1	-	-	-	-	2
10-19	1	-	2	-	-	-	-	-	3
20-29	4	1	-	-	-	-	-	-	5
30-39	1	-	-	1	-	-	-	-	2
40-49	3	-	-	-	1	-	-	-	4
50-59	5	2	-	3	2	-	-	-	12
60-69	1	-	-	-	-	-	-	-	1
Totals for First 100 Years	25	3	4	7	3	1	0	0	43
70-79	6	6	5	3	1	2	-	-	23
80-89	105	1	5	5	1	1	-	1	119
90-99	175	-	1	6	2	-	1	-	185
1900-09	51	3	5	6	2	-	1	-	68
10-19	23	10	11	12	2	3	2	-	63
20-29	9	8	3	7	3	2	-	-	32
30-39	17	6	5	6	1	-	-	-	35
40-49	19	5	8	4	1	-	-	-	37
50-59	9	-	15	7	10	-	-	-	41
1960-69	20	3	12	12	1	-	-	-	48
Totals for Second 100 Years	434	42	70	68	24	8	4	1	651
Totals for 200 Years	459	45	74	75	27	9	4	1	694

Bollinger was not able to establish a stable relationship for any of the subareas of the southeastern United States seismic region. A recurrence curve for the southern Appalachian seismic zone must, therefore, be extrapolated from the relationship developed for the larger region if consideration is to be given to the fact that this area has been seismically more active than the average of the larger region. It is assumed that the recurrence curve for the southern Appalachian seismic zone should predict slightly higher recurrence rates than the curve for the combined region. Bollinger's relationship for the recurrence curve given in

Equation 1-2 for the larger region is actually stated with standard deviation limits as follows:

$$\log N = (3.01 \pm 0.04) - (0.59 \pm 0.02) I_o \quad \text{for } V \leq I_o \leq VIII \quad (1-4)$$

This equation when applied to the southern Appalachian seismic zone would provide the following equations in terms of Mercalli Intensities and Richter Magnitudes:

$$\log N = 2.72 - 0.57 I_o \quad (1-5)$$

$$\log N = 3.58 - 0.86 M \quad (1-6)$$

Equation 1-5 will be used here for the seismic

hazard assessment of the southern Appalachian seismic zone (area = 62,500 sq mi).

1.2.4 MAXIMUM POSSIBLE EARTHQUAKE

The maximum possible earthquake for a given region is estimated from:

- a. The seismic history of the relevant fault zones, seismic zones, and host tectonic province
- b. Geologic and geomorphic evidence of recent fault movements, e.g., offset strata, scarps, scarp-like features
- c. Comparison of the seismic history of the zones and province with similar structural and tectonic environments elsewhere
- d. Empirical fault length-fault displacement-magnitude relationships (e.g., Bonilla and Buchanan, 1970)

For the conditions described for the southern Appalachian seismic zone, earthquakes cannot be associated with known tectonic structures. The SSE selected for the design of nuclear plants located in the seismic zone should be the maximum possible earthquake according to the provisions of 10 CFR 100, Appendix A.

These procedures require careful consideration of the local and regional geology and of the seismic history, and in the final analysis, the epicentral intensity of the SSE is based on the collective judgment of experienced geologists, seismologists, and engineers.

Nuclear power plants located in the southern Appalachian seismic zone are currently designed for an SSE having a Mercalli Intensity of VIII. The two largest earthquakes on record in this zone (see Table 1-2) had epicentral intensities between VII and VIII. An Intensity VIII will therefore be used as the cutoff intensity in the probability calculations.

1.2.5 INTENSITY ATTENUATION RELATIONSHIPS

The rate at which the strength of earthquake ground motion attenuates with distance from the epicenter, or causative fault, is required for seismic hazard analyses. Prior to 1933, all data on the intensity of strong earthquake ground motion throughout the United States were expressed in terms of Rossi-Forel or Mercalli Intensities. Since 1933 a limited number of strong motion accelerograph records have become available in California and other

TABLE 1-2. CHRONOLOGICAL LISTING OF EARTHQUAKES WITH $I_0 > VII$ IN THE SOUTHEASTERN UNITED STATES SINCE 1870 (Bollinger, 1973)

Date	Intensity	State	Seismic Zone
Feb 10, 1874	V-VII	North Carolina	Southern Appalachian
Dec 22, 1875	VII	Virginia	Central Virginia
Aug 31, 1886	X	South Carolina	South Carolina-Georgia
Oct 22, 1886	VII	South Carolina	South Carolina-Georgia
May 31, 1897	VIII	Virginia	Southern Appalachian
Jan 27, 1905	VII-VIII	Alabama	Southern Appalachian
Jun 12, 1912	VI-VII	South Carolina	South Carolina-Georgia
Jan 1, 1913	VII-VIII	South Carolina	South Carolina-Georgia
Mar 28, 1913	VII	Tennessee	Southern Appalachian
Feb 21, 1916	VI-VII	North Carolina	Southern Appalachian
Oct 18, 1916	VII	Alabama	Southern Appalachian
Jul 8, 1926	VI-VII	North Carolina	Southern Appalachian
Nov 2, 1928	VI-VII	North Carolina	Southern Appalachian

nearby West Coast states. However, such data are practically nonexistent in states east of the Mississippi. Data on earthquake intensity and the attenuation rate for the southeastern United States region are, therefore, expressed in terms of Mercalli Intensities. (For descriptions and comparisons of the Rossi-Forel and Mercalli Intensity scales, see Chapter 6 below).

The focal depth of the earthquake and the near surface soil and geological conditions have a significant influence on the intensity-attenuation relationship. Apparently surface faulting does not occur for earthquakes in the eastern United States, and the center of energy release is deeper than the average for California earthquakes. The earthquake energy is therefore radiated over a larger area than is experienced in California, and California data cannot be extrapolated directly to this region. The procedure that must be used, therefore, is to develop intensity-attenuation relationships from past earthquakes in the seismic zone of interest. This presents a problem since there are few earthquakes within the southern Appalachian seismic zone for which there are documented attenuation data. Some earthquakes will, therefore, need to be considered from the larger southeastern United States seismic region. Since within a specific region, some variation in radiation pattern must be expected solely from differences in focal depths, types of faulting, transmission paths, etc., it is evident

that the intensity attenuation relationship is a probabilistic problem within itself. The procedure followed here is to use an attenuation relationship that represents an approximate average of seven earthquakes occurring in the southeastern United States, as listed in Table 1-3. Following a brief discussion of these earthquakes, Figure 1-2 compares the resulting normalized attenuation curves.

The Charleston earthquake of 1886 is significant for two reasons: first, it was a major earthquake in the southeastern United States seismic region for which the intensity attenuation relationships have been well documented; second, it is the only major earthquake in this region in the past 300 years. According to Bollinger (1972) there were no previous severe earthquake shocks in the Charleston area for 200 years prior to this earthquake. Because of this long aseismic history, this event is often cited as an example of why no region in the United States can be considered completely safe from the hazard of severe earthquake ground motion.

The Giles County earthquake of 1897 and the Elgood, West Virginia, earthquake of 1969 are important because their epicenters were located within the Valley and Ridge province. The Giles County earthquake had an epicentral intensity of VII-VIII and the Elgood earthquake had an epicentral intensity of V.

TABLE 1-3. EARTHQUAKES CONSIDERED IN INTENSITY ATTENUATION STUDY FOR SOUTHERN APPALACHIAN SEISMIC ZONE

No. (See Fig. 1-2)	Date	Location	Epicentral Intensity
8	Aug 31, 1886	Charleston, South Carolina	IX-X
2	May 31, 1897	Giles County, Virginia	VII-VIII
1	Nov 20, 1969	Elgood, West Virginia	V
3	Feb 21, 1916	Waynesville, North Carolina	VI-VII
4	Nov 2, 1928	Asheville, North Carolina	V-VI
5	Dec 22, 1875	Chesterfield County, Virginia	VI-VII
7	Jan 1, 1913	Union County, South Carolina	VII-VIII
6	Oct 18, 1916	Birmingham, Alabama	VI-VII

FIGURE 1-2. COMPARISON OF NORMALIZED INTENSITY ATTENUATION CURVES FOR EARTHQUAKES GIVEN IN TABLE 1-3

The radiation patterns of the Giles County and Elgood earthquakes were nonuniform, as the highly complex local geology apparently had a significant effect. There was a strong transmission of energy approximately parallel to the longitudinal axis of the Valley and Ridge province for the Giles County earthquake, but strong motion was also experienced to the east. The radiation pattern of the Elgood earthquake was equally complex and exhibits a so-called "halo effect" in that there are two other isolated areas north and south of the epicentral area, having an intensity of IV. This has been noted for other earthquakes in this region (Bollinger and Hopper, 1969), and is apparently associated with the highly folded and faulted geology of the region.

Intensity attenuation curves for the Giles County and Elgood earthquakes were prepared, and normalized curves (1 and 2) are compared with the Charleston attenuation curve in Figure 1-2. Both curves lie above the Charleston curve from I_0 to nearly $(I_0 - II)$. For lower intensities, the attenuation rate of the Giles County earthquake is more rapid. The fact that attenuation curves for these two earthquakes lie above the Charleston curve and then attenuate more rapidly probably implies that the two smaller earthquakes occurred at geometrically shallower depths.

The epicenters of the North Carolina earthquakes at Waynesville (1916) and Asheville (1928) were located in the Blue Ridge province near the center of the southern Appalachian seismic zone (very close to the Tennessee-North Carolina border). The former had an estimated epicentral intensity of VI to VII and the latter V to VI, both on the Rossi-Forel scale. There was strong radiation of energy in the northeast-southwest direction in the 1916 earthquake. The pattern for the smaller 1928 earthquake was somewhat distorted, having a northeast-southwest trend in the Ridge and Valley province but a much stronger radiation pattern to the southeast in the Coastal Range province. In contrast, very little energy was transmitted to the northwest normal to the longitudinal axes of the Valley and Ridge, and Blue Ridge provinces. The intensity attenuation curves for these two earthquakes were prepared, and normalized curves (3 and 4) are compared in Figure 1-2. The 1916 Waynesville earthquakes compare well with the normalized Charleston curve, but the 1928 Asheville earthquake shows a more rapid attenuation rate that apparently results from the lack of energy radiation to the northwest of the epicenter for this earthquake.

Three additional earthquakes that were more remote from the southern Appalachian seismic zone are given to obtain some idea of

the intensity attenuation characteristics in other geologic provinces and in other parts of the southeastern United States seismic region, and also to provide a larger sample of curves. The epicenters of the Chesterfield County (1875) and the Union County (1913) earthquakes were located in the Piedmont province. The Birmingham earthquake of 1916 had its epicenter near the southwest extremity of the Valley and Ridge province. Intensity attenuation curves were prepared, and normalized curves are compared in Figure 1-2. The attenuation curve for the 1875 Virginia earthquake has the general shape of the curve for the Giles County earthquake but the attenuation rate is more rapid. The attenuation curve for the 1916 Alabama earthquake has the general shape of the curve for the Waynesville earthquake, but again the attenuation rate is more rapid. The Union County earthquake is unique in that it departs completely from all other curves in this region and demonstrates a very rapid rate of attenuation. This earthquake was noted by Taber (1913) for its abnormally small felt area and he postulates that it perhaps resulted from a very shallow earthquake of limited fault length. (Because of its anomalous nature it will not be considered further in this study.)

In summary, three of the earthquakes studied, the Elgood, the Waynesville, and the Birmingham earthquakes, have intensity attenuation curves with the same general characteristic shape as the Charleston curve. Three other earthquakes, the Giles County, the Asheville, and the Chesterfield County earthquakes, have intensity attenuation curves that exhibit lower close-in attenuation but a higher rate at the lower intensity levels than the Charleston normalized curve. These may represent geometrically shallower depth earthquakes. Since earthquakes having intensity attenuation curves of both shape characteristics have occurred throughout the region, the approximate average

of the six curves was adopted for earthquakes having epicentral intensities of VI and less. A comparison of this mean curve with the normalized Charleston attenuation curve is also provided in Figure 1-2. The normalized Charleston attenuation curve was used for epicentral intensities of VIII and greater. An average of the Charleston curve and the average for the six smaller earthquakes was used as an attenuation curve for earthquakes having an epicentral intensity of VII. The difference between the mean of the six smaller earthquakes and the Charleston curve in Figure 1-2 is slight and either could have been used for all epicentral intensity levels with little effect on the final seismic risk assessment. This refinement was therefore probably not justified. The intensity attenuation curves used in the seismic risk analysis are given in Figure 1-3.

1.2.6 PROBABILITY CALCULATIONS

Discussion of the seismic history (Sec. 1.2.2) indicated that past earthquakes in the southeastern United States seismic region were independent of the geologic provinces and probably not associated with the exposed geologic structures. The southern Appalachian seismic zone, which is a part of this larger region, relatively is seismically homogeneous. Based on the nearly uniform distribution of past earthquake epicenters in this zone, the assumption that a given seismic event has an equal probability of occurring anywhere in the seismic zone is reasonably well justified. Since only earthquakes with epicentral intensities of V or greater were considered in the development of the basic recurrence relationship for the larger southeastern United States seismic region, relatively few of the Charleston 1886 aftershocks were considered. There has been no other major seismic event producing significant aftershocks included in the data. Therefore, the assumption that the arrival rates of all

FIGURE 1-3. INTENSITY ATTENUATION CURVES FOR SEISMIC RISK ANALYSIS OF SOUTHERN APPALACHIAN SEISMIC ZONE FIRM SITE CONDITIONS

earthquake events in the region are independent and are Poisson distributed, is reasonably well satisfied. Thus, the procedure developed by Housner (1969), which is based on these uniform seismicity assumptions, can justifiably be used for the probability calculations. A discussion of the mathematical relationships upon which the calculations are based is provided in AA (1976b). The recurrence relation given in Equation 1-5, and the intensity attenuation curves given in Figure 1-3 were used in the calculations. The maximum epicentral intensity considered was Mercalli Intensity VIII. The resulting probabilities are given in Table 1-4. The probability of an Intensity VIII seismic environment being equaled or exceeded is

estimated to be about 0.015% for a period (service life) of fifty years (see AA, 1976b). The calculations assume firm site conditions.

1.2.7 ASSESSMENT OF RESULTS

It is essential that the major factors influencing the results of the seismic risk analysis be carefully reviewed to ascertain the relative conservatism with which each factor has been treated and the probable effects on the overall results. There are three major factors that require consideration: (1) the frequency of earthquake recurrence relationship, (2) the intensity attenuation relationship, and (3) the maximum epicentral intensity considered potentially possible in the southern Appalachian seismic zone.

The frequency of recurrence of earthquakes in the southern Appalachian seismic zone has been defined by Equation 1-5, which is a mean plus σ curve defined by Bollinger for the total southeastern United States seismic region. This curve was used as the seismic history indicates that the southern Appalachian seismic zone has been more active than the average of the larger region. The recurrence curve based on the observed seismic history of the smaller zone for the past 100 years indicates a recurrence rate of about 50 years for an Intensity VIII as compared to the 69 years given by the

TABLE 1-4. PROBABILITIES IN PERCENT OF EXPERIENCING DIFFERENT LEVELS OF MERCALLI INTENSITY AT ANY INDIVIDUAL SITE IN THE SOUTHERN APPALACHIAN SEISMIC ZONE. FIRM SITE CONDITIONS, MAX I_0 = VIII

Mercalli Intensity	Service Life, in Years			
	10	25	50	100
VIII	< 0.1	< 0.1	< 0.1	< 0.1
VII	0.8	2.1	4.1	8.0
VI	14.1	31.7	53.3	78.2
V	60.1	87.0	98.0	100.0

statistically derived Equation 1-5. The recurrence rate for Intensity VII is about the same but for Intensity VI the recurrence rate is increased from 50 years to about 69 years. Therefore, if this curve had been used for the probability calculations rather than Equation 1-5, the probability of experiencing an Intensity VIII environment at a specific site in the southern Appalachian seismic zone would have been increased by about 40%. The probability, however, would still be about 0.02% for a 50-year period (service life). The probability of experiencing Intensity VI and VII environments would be increased by a lesser amount. At Intensity V, there would be little change in probability.

The record of lower intensity earthquakes is known to be incomplete in this region because of low population density and because of inadequate instrumentation. Also, the period of records is too short to establish an adequate recurrence rate for the higher intensities. Therefore, the recurrence curve has been based on data from a larger region, and a correction has been made for the fact that the southern Appalachian seismic zone is more active seismically than the average of the larger region. Equation 1-5 is therefore recommended as a reasonable average relationship for the southern Appalachian seismic zone. Potential variations in probability percentages due to possible errors in this relationship are estimated to be not more than $\pm 10\%$ from those given in Table 1-4 (i.e., 0.9 to 1.1 times the percentages indicated in the table).

Average intensity attenuation curves were used in the probability calculations. Actually these curves are influenced by many variables and vary from one earthquake to the next. The statistical variation in the attenuation curves could be considered in the probability

calculation, but, in truth, a much larger sample of records is needed. There may also be some error in estimating areas from small scale iso-seismal maps, and there is some error in converting from the Rossi-Forel to the Mercalli scale. It is estimated that the inclusion of the variation in intensity attenuation as a variable in the probability calculations would increase the probability percentages given in Table 1-4 by not more than 5%. Possible errors in the computed average intensity curves are estimated to be roughly about $\pm 10\%$.

The maximum potential epicentral intensity (Intensity VIII) considered in the probability calculation is the most important variable in the seismic risk study, as it has by far the greatest impact on the results. Since an Intensity VII to VIII has been experienced in the southern Appalachian seismic zone, the selection of Intensity VIII would appear to be a reasonable estimate for the SSE. The only question is whether a higher intensity should have been considered in the probability calculations. A higher intensity has been experienced only once in 300 years in the entire southeastern United States seismic region, but this was in Charleston, which is not in the southern Appalachian seismic zone. Could a similar earthquake occur elsewhere in the southeastern United States, or is this restricted to the Charleston area because of local geologic structures? To assess the impact of this value on the seismic risk analysis, calculations have also been performed using an Intensity IX as a cutoff value rather than Intensity VIII. The results indicate that the probability of experiencing an Intensity VIII environment is increased from 0.015% to 0.8%. Therefore, the intensity assigned to the SSE should be Intensity VIII, and the probability of experiencing an Intensity VIII environment under any reasonable set of assumptions in a fifty year period is less than 1%.

1.2.8. CONVERSION OF INTENSITY TO PEAK GROUND ACCELERATION

Probability calculations are frequently expressed in terms of peak ground acceleration. Only in California have sufficient strong motion acceleration records been collected to provide a direct attenuation relationship for acceleration with distance. In all other areas in the United States, Mercalli Intensities are converted to "equivalent" peak ground acceleration and a direct substitution is then made in the calculations, such as those listed in Table 1-4. There are more than a dozen published correlations between intensity and peak ground acceleration (see Chapter 7 below), and the range in answers using the different conversion relationships is more than an order of magnitude. The reason is that Mercalli Intensities are a function of many variables other than peak ground acceleration. For example, Mercalli Intensity is a subjective rating system based primarily on the reactions of individuals during an earthquake, and upon the observed damage to the ground, embankment slopes, buildings, and other man-made structures. The structural damage experienced is strongly influenced by the quality and type of construction, the duration and frequency content of the ground motion, the subsurface soil condition, and the strength of motion. The duration of the strong motion is influenced by the magnitude of the earthquake, and the frequency content of the ground motion is influenced by distance from the causative fault, subsurface soil conditions, and probably earthquake magnitude and type of faulting. For these reasons, Mercalli Intensities, which are defined by all of these variables and are the result of field observations, can be assessed in a seismic risk analysis with a reasonable level of confidence, but peak ground accelerations cannot be assigned to the different intensity levels with the same level of confidence, as peak acceleration is only one of many

factors that determine intensity. In this sample study, the seismic risk has been expressed in terms of Mercalli Intensity. The problems of converting Mercalli Intensity to peak ground acceleration are discussed more fully in AA (1976b). If Mercalli Intensities are converted to peak ground accelerations, the variations in the conversion relationships should be considered probabilistically.

1.3 VARIABLE SEISMICITY APPROACH

The seismic hazard assessment of a site can be made using a variable seismicity approach (Cornell, 1970). This approach is required for sites where historic earthquakes of greatest magnitude can be correlated with known tectonic structures, or are identified with tectonic provinces in which the site is not located. The approach used is based on a line source model developed by Der-Kiureghian and Ang (1975). The primary feature of the method is that variations in the seismicity of the region due to one or more predominantly active fault areas can be considered. The method superimposes the effects of postulated earthquakes from each source to develop probability estimates of ground motion intensity at a particular site within the region being investigated. It is assumed that occurrences of earthquakes from a particular fault source can be modeled by a homogeneous Poisson process with a constant average rate per year.

Except for the probability calculations, the procedure involves the same basic steps discussed for the uniform seismicity approach. To demonstrate the procedure, a hypothetical site has been selected in western Tennessee, approximately 20 mi north of Jackson, Tennessee (Fig. 1-4). This site has been selected because of its close proximity to the New Madrid, Missouri, seismic zone, and because seismic and geologic information was readily available for this region. Also, this seismic zone has been

FIGURE 1-4. MAP SHOWING LOCATION OF HYPOTHETICAL SITE A

the source of some of the most severe earthquakes experienced in the United States. The hypothetical site will be designated Site A.

1.3.1 REGIONAL GEOLOGY

The geologic history of interest to this study, along with fault descriptions, is given by TVA (1972). The topography in eastern Tennessee at the end of the Paleozoic era (about 300 million years ago) sloped gently to the southeast from the Ozark Highlands in the north to central Tennessee. The area had been uplifted such that the underlying Paleozoic rocks formed a huge arch. At one time, more than 8000 ft of Paleozoic rock (from the Pennsylvanian to the Ordovician age, deposited 300 to 600 million years ago) covered the arch. This material was gradually eroded away during the Cretaceous period of the Mesozoic era (100 million years ago) leaving exposed Cambrian

Age rocks in northwest Tennessee. The eroded debris accumulated to the east to form the Cretaceous Tuscaloosa gravels.

In late Cretaceous time (70 to 100 million years ago), the sea advanced northward across the gradually subsiding arch as downwarping of this region began. Between late Cretaceous and Eocene time, the region known as the Pascola Arch was depressed about 4000 ft; from 1000 ft above sea level to about 3000 ft below, and was covered with unconsolidated sands and clays. It will be noted that the depressed Pascola Arch connects the structures that are today identified as the Ozark Dome in the northwest with the Nashville Dome in the southeast. The submerged area in the Cretaceous to Eocene time is referred to as the Mississippi Embayment. The Paleozoic rocks overlying the basement rock in the upper Mississippi Embayment area are about 7000 ft in thickness. In

southwestern Tennessee, the depth to basement rock is about 10,000 ft. It is significant that major faults are indicated in the basement rock at the same general locations as known faults in the overlying rock formations. These include the New Madrid-Big Creek, Cottage Grove, Rough Creek, western Kentucky, and the Ste. Genevieve fault systems.

The New Madrid fault system extends to the southwest from near Vincennes, Indiana, for a distance of more than 200 mi through southeastern Illinois, southeastern Missouri, and into northeastern Arkansas. Outside the limits of the Mississippi Embayment, its location is well defined by surface mapped faults in exposed Paleozoic rocks. Within the Mississippi Embayment area, its location is generally inferred from epicenters of recent earthquakes (1811 to date) and from some borings and limited mapping. The western edge is defined by Fuller's epicentral line of the 1811-12 earthquakes. The eastern edge is defined by the extension of the Big Creek fault to the northeast, an area sometimes referred to as the Big Creek-Jackson Purchase faulted area.

The western Kentucky faulted area could be considered a northeastern extension of the Big Creek-Jackson Purchase faulted area that lies outside the Mississippi Embayment area. (However, it does not share the same recent seismic activity). This area extends about 100 mi eastward from the Mississippi Embayment boundary. It is bounded on the northwest by the New Madrid fault system and on the north by the Rough Creek fault.

The Rough Creek-Cottage Grove fault system is an east-west fault system that extends across western Kentucky and southern Illinois. The Rough Creek fault extends east from its intersection with the New Madrid fault system. The Cottage Grove fault extends west from this intersection.

The Ste. Genevieve fault system extends southeastward from near St. Louis, Missouri, across southern Illinois until it intersects the New Madrid fault system. It then extends southeast as a buried fault crossing the entire New Madrid, Big Creek-Jackson Purchase fault system.

The faulting most important to the seismic risk study of the Site A area is identified with the New Madrid-Jackson Purchase fault system. Faults in this area are directly associated with the subsidence that produced the Mississippi Embayment. Basic igneous intrusive rocks have been detected in this area, which indicates that faulting has occurred back to Paleozoic time. The discussion of the area's seismic history shows that this zone remains one of the most active in the midcontinent and defines the seismic risk to Site A.

1.3.2 SEISMIC HISTORY

Global seismicity studies indicate that the interior of continental land masses are generally seismically stable. Central United States has behaved in this characteristic fashion for more than 150 years. However, in 1811-12 the upper Mississippi Embayment area was subjected to some of the strongest earthquake ground motions experienced in the recorded history of the United States. In less than three months during this period, three separate earthquakes, each having local magnitudes greater than 7, occurred in this area. Also, more than 1800 aftershocks were associated with these three major earthquakes (Fuller, 1912). These major earthquakes are referred to as the New Madrid earthquakes, for their epicenters were located along a fault line passing near New Madrid, Missouri. These events are significant to the seismic hazard study of Site A, even though of low order of recurrence.

The first of the three major New Madrid shocks occurred on December 16, 1811 and has been estimated (Nuttli, 1973) to have had a local magnitude of 7.5.* This was followed by nearly 500 aftershocks before the second major shock of local magnitude 7.3 occurred on January 23, 1812. This was followed by a local magnitude 7.8 shock on February 7, 1812. Between December 16 and March 15, over 1800 total shocks were recorded by Brooks in Louisville, Kentucky (Fuller, 1912). The total energy release by the entire series of major and minor shocks has been estimated to have been equivalent to one local magnitude 8 earthquake. On the basis of three separate major shocks, or on the basis of total energy release, this series of seismic events represents one of the most severe that has occurred in modern history in the United States, including West Coast areas. However, these events are of relatively infrequent occurrence.

The magnitudes of the three major New Madrid earthquakes indicate that each would have been associated with a major displacement along a fault length of 100 mi or more. Because much of the region is underlain by deep, unconsolidated, saturated river deposits, broad areas experienced severe damage. Lakes were uplifted and drained, and in other areas sand boils erupted and areas were flooded with water several feet deep. Damaging ground motion (Mercalli Intensity VII) extended over an area estimated to have been more than 200,000 sq mi, and ground motion severe enough to alarm the general population (Intensity V) was felt as far away as Detroit and Washington, DC (Nuttli, 1973).

*Nuttli estimates these earthquakes in terms of body wave magnitude m_b . These events have been converted here to local magnitudes, M , in accordance with Richter (1958).

1.3.3 EARTHQUAKE FREQUENCY OF RECURRENCE RELATIONSHIPS

A frequency recurrence relationship for earthquakes, expressed either in magnitude or epicentral intensity, is required for the seismically active areas contributing the seismic risk to Site A. However, it is desirable to start with relationships that apply to a larger region so that the general seismicity is defined first. These relationships can then be extrapolated to the smaller zones and areas. Magnitude recurrence relationships have been developed for the Central Mississippi Valley by Nuttli (1974). For this area (nearly 100,000 sq mi) and using data since 1833, Nuttli obtained an earthquake magnitude frequency of recurrence relationship by a weighted least squares procedure expressed as:

$$\log N = 3.76 - 0.92 m_b \quad (1-7)$$

where m_b is the earthquake body wave magnitude and N is the number of earthquakes per year occurring in magnitude interval $m_b \pm 0.2$. The year 1833 was selected so that there would be little influence from the aftershocks of the 1811-12 earthquakes, and by 1833, the Central Mississippi and the Lower Ohio River Valleys were populated enough that earthquakes would be observed and reported. However, the data are far from complete until after 1870. In testing the data by Stepp's procedures, Nuttli found that the data for earthquake magnitudes $m_b \leq 3.3$ are incomplete for the entire period of record, i.e., 1833 to 1972. This results because an extensive instrumentation network is required to detect such low energy level earthquakes. The data for earthquake magnitudes $3.4 \leq m_b \leq 3.8$ were found to be adequately recorded for only the last 50 to 70 years of the record. For magnitudes $3.9 \leq m_b \leq 4.3$, the data appeared to be complete for the past 100 years (since 1872). According to Nuttli, for $m_b \geq 3.9$, the energy release is sufficiently large and the population density adequate to

detect and report nearly all such earthquakes. However, the analysis also shows that earthquakes larger than $m_b \geq 4.9$ have been insufficient in occurrence, even though reported, to reliably define the curve. In other words a longer period of records is needed. With this background, Nuttli developed Equation 1-7 by a least squares procedure which gave more weight to earthquake magnitudes less than 4.6 than for those above this value, which is a method contrary to those of other investigators (see Couch et al., 1972; Stepp, 1972).

In order to provide a relationship that gives greater consideration to more recent data, and to earthquake magnitudes equal to 4.6 and greater, only the data from 1870 through 1969 is considered to derive the following useful equation:

$$\text{Log}N = 4.69 - 1.09 m_b \quad (1-8)$$

For the 10,000 sq mi area that encompasses the upper Mississippi Embayment area, the following two equations can represent upper (Eq. 1-9) and lower (Eq. 1-10) bound recurrence relationships for predicting larger magnitude earthquakes there:

$$\text{Log}N = 4.34 - 1.09 m_b \quad (1-9)$$

$$\text{Log}N = 3.50 - 0.92 m_b \quad (1-10)$$

Here, the recurrence rate for $m_b = 7$ varies from once in 865 years for the upper bound curve to once in 1940 years for the lower bound

curve. Thus, a single shock of the New Madrid earthquake series in 1811-12 has an average recurrence rate of about 1000 to 2000 years when these recurrence curves are used. It should also be noted that the recurrence rate for an $m_b = 7$ earthquake for the Central Mississippi Valley area varies from 430 to 870 years, which implies that large magnitude earthquakes can occur outside the New Madrid seismic zone. However, this may not be physically possible, which points out the uncertainty in predicting large magnitude earthquakes. It should be recognized that the accuracy of this prediction is subject to considerable question as it is based on a very short period of records. The best that can be stated is that an $m_b = 7$ earthquake has a recurrence rate of 500 to 2000 years in the upper Mississippi Embayment area.

1.3.4 INTENSITY ATTENUATION RELATIONSHIPS

All local data for the Mississippi Embayment area are in terms of Mercalli Intensity, and the energy radiation pattern in this area is significantly different than has been experienced in California. Therefore, California attenuation data cannot be extrapolated to this area. To develop intensity attenuation relationships for this area, the radiation patterns of significant earthquakes have been considered (Table 1-5).

TABLE 1-5. EARTHQUAKES CONSIDERED IN INTENSITY ATTENUATION STUDY FOR SITE A

Date	Location	Epicentral Intensity	Estimated* Magnitude
Dec 16, 1811	New Madrid, Missouri	XI	7.2
Oct 31, 1895	Charleston, Missouri	IX	6.2
Nov 9, 1968	Hamilton Co., Illinois	VII	5.5
Apr 19, 1917	Ste. Genevieve, Missouri	VI	5.0
Mar 3, 1963	Poplar Bluff, Missouri	VI	4.7

*From Nuttli (1973, 1974).

The procedure followed in computing the intensity attenuation relationships for these earthquakes used the appropriate isoseismal maps (see AA, 1976b). The total area inside each isoseismal line was first computed. The radius of a circle having an equivalent area was then computed and intensity plotted in Figure 1-5. Following a brief discussion of the five earthquakes of Table 1-5, Figure 1-6 shows the intensity attenuation curves for firm site conditions for the Central Mississippi Valley, to be used in the probability calculations.

FIGURE 1-5. COMPARISON OF EARTHQUAKE INTENSITY ATTENUATION CURVES

Intensity attenuation relationships have been developed for the New Madrid earthquake of December 16, 1811. A relatively small area in the Mississippi River lowlands was assigned a Mercalli Intensity of XI by Nuttli (1973). The

FIGURE 1-6. INTENSITY ATTENUATION CURVES FOR CENTRAL MISSISSIPPI VALLEY EARTHQUAKES - FIRM SITE CONDITIONS

attenuation curve attributed to Nuttli with an epicentral intensity XI is shown in Figure 1-5. This curve is also modified in Figure 1-5 to an epicentral intensity of X, which would be more typical for this earthquake for firm site conditions.

The 1895 earthquake, which had its epicenter in the Mississippi Embayment area near Charleston, Missouri, is the largest earthquake in central United States since the 1811-12 New Madrid earthquakes. An epicentral intensity of IX was assigned to this earthquake (Nuttli, 1974). The intensity attenuation curve is shown in Figure 1-5 for an epicentral intensity of about IX. This curve has also been adjusted to an epicentral intensity of VIII which would be more typical of firm site conditions.

The Hamilton County earthquake of 1968 occurred near the northern extension of the New Madrid fault system, but outside the Mississippi Embayment area, and was the largest earthquake in central United States since the 1895 earthquake. This region is underlain by about 13,000 ft of Paleozoic rock and represents firm site conditions. The intensity attenuation curve for this earthquake has been plotted in Figure 1-5 with no epicentral intensity adjustment.

The earthquake of 1917 had its epicenter near Ste. Genevieve, Missouri, which places it well out of the Mississippi Embayment area in the vicinity of the Ste. Genevieve fault. Because of the lower magnitude of this earthquake, and because of more firm subsurface conditions, the intensity attenuation curve for this event has been plotted in Figure 1-5 with no epicentral intensity adjustment.

Points on the intensity attenuation curve for the Poplar Bluff earthquake of 1963 are shown in Figure 1-5 to indicate the close agreement with the attenuation curve for the Ste. Genevieve earthquake. This earthquake had its epicenter about 20 mi northwest of the New Madrid fault line but within the Mississippi Embayment area. Depth of unconsolidated sediments should be relatively shallow in this area. Also, the strength and duration of epicentral ground motion would ordinarily not have been great enough to cause severe foundation damage. Therefore, no adjustment was made in the epicentral intensity when plotting the attenuation relationship.

1.3.5 MAXIMUM POSSIBLE EARTHQUAKE

The maximum earthquake magnitude considered physically possible must be established to provide meaningful probability calculations. The New Madrid 1811 earthquakes provide excellent data for such a prediction, for in the

short period of three months, three earthquakes with body wave magnitudes, m_b , varying from 7.1 to 7.4 (local magnitude, $M = 7.3$ to 7.8) occurred along the New Madrid fault system. The three earthquakes represented a combined energy release of about $m_b = 7.5$. Therefore, the possibility of a magnitude m_b of 7.5, or greater, event must be considered. Probability procedures provide an excellent method of assessing the impact of this variable. In the probability calculations, maximum earthquakes of magnitudes m_b of 7.0, 7.5, and 8.0 will be considered.

1.3.6 PROBABILITY CALCULATIONS

In the line-source model developed by Der-Kiureghian and Ang (1975), it is assumed that the earthquakes originate as slips along geologic faults and that the intensity of the motion at the site of interest is primarily contributed by the energy released in the length of the fault closest to the site. For this analysis, the orientation of the causative faults and their approximate locations are known. The general location and approximate fault orientations are shown in Figure 1-7. Historical earthquakes were grouped into the source areas indicated in the figure and each area numbered. The number of earthquakes occurring in each area is indicated in parentheses. Each area is small enough that the distance of each epicenter within the area to Site A can be assumed to be a constant. The number of occurrences per year in each area were then adjusted systematically to agree with the recurrence curves given by Equations 1-9 and 1-10. A mathematical fit to the attenuation curves in Figure 1-6 between distances of 30 and 100 mi was then provided, and separate probabilities computed for Site A for each source area. Body wave magnitudes of 7.0 and 7.5 were considered for the lower bound curve, and 7.5 and 8.0 were considered for the upper bound

FIGURE 1-7. IDEALIZATION OF SOURCES FOR SITE A

curve. These are equivalent to local magnitudes, M , of 7.1 to 7.9 and 7.9 to 8.7, respectively. The results are listed in Table 1-6. Limiting curves for a service life of 50 years are provided in Figure 1-8.

1.3.7 ASSESSMENT OF RESULTS

The seismic risk analysis for Site A is affected by three major factors: (1) the maximum earthquake considered potentially possible,

(2) the frequency of earthquake recurrence relationship, and (3) the intensity attenuation relationship.

In Figure 1-8, it is evident that the maximum potential earthquake and the recurrence curve used both have a significant impact on the results. The maximum potential earthquake has its greatest influence for intensities at Site A greater than about VII⁺. Below Intensity VII the effect is minimal. Based on world

TABLE 1-6. PROBABILITIES IN PERCENT OF EXPERIENCING DIFFERENT LEVELS OF MERCALLI INTENSITY AT SITE A FOR FIRM SITE CONDITIONS

Log N(m) = 4.34 - 1.09m $m_u = 7.0$ to 7.5						Log N(m) = 3.5 - 0.92m $m_u = 7.5$ to 8.0				
MM Intensity	Return Period, Years	Service Life, Years				Return Period, Years	Service Life, Years			
		10	25	50	100		10	25	50	100
V	15	50	89	97	99	15	50	89	97	99.9
V + 1/2	38 - 39	23	49	74	93	32	27	55	80	96
VI	92 - 95	10	24	42	66	67 - 68	14	31	53	78
VI + 1/2	207 - 223	5	11	21	37	131 - 136	7	17	31	53
VII	442 - 527	2	5	10	18	246 - 263	4	9	18	33
VII + 1/2	920 - 1390	1	2	4	9	444 - 503	2	5	10	19
VIII	1900 - 6000	0.4	0.8	2	3	776 - 971	1	3	6	11
VIII + 1/2	4350 - 125000	0.1	0.3	0.6	1	1330 - 2030	0.6	2	3	6
IX	12300 - ∞	< 0.1	0.1	0.2	0.4	2300 - 5600	0.3	0.8	2	3

FIGURE 1-8. COMPARISON OF MERCALLI INTENSITY PROBABILITY RELATIONSHIPS FOR SITE A - FIRM SITE CONDITIONS

wide seismicity as well as local seismic history, a maximum potential earthquake $m_b = 7.5$ ($M = 8.7$) is unreasonably high. Similarly, a maximum potential value of $m_b = 7.0$ ($M = 7.1$) would be unreasonably low, as values greater than this are a matter of historical record. However, this analysis illustrates clearly the

impact of a more, or less, conservative decision.

The recurrence curves have a nearly constant effect on the probability calculations for all intensities greater than about VI. At Intensity IX, the difference in effect is about a

factor of 2, increasing the probability percentage from 0.4 to 0.9 for a maximum potential earthquake of $m_b = 7.5$ ($M = 7.9$). The recurrence curve $\text{Log}N = 4.34 - 1.09 m_b$ is the more logical choice, but the impact of the more conservative equation $\text{Log}N = 3.5 - 0.92 m_b$ is again clearly evident.

Even though 10 CFR 100, Appendix A does not indicate that the maximum potential earthquake affecting the site (SSE) should be selected by probabilistic procedures, it is evident that a seismic risk study can provide useful supplementary information when making this decision. On the basis of the curves given in Figure 1-8, the best estimate of the probability of experiencing an Intensity IX environment at Site A is about 0.4% for a service life of 50 years. A more conservative recurrence curve would increase this value by a factor of about 2, to 0.9%. Adding an additional overly conservative maximum potential earthquake assumption, $m_b = 8.0$ ($M = 8.7$), increases the probability by an additional factor of 2, to 2%. Although the U.S. Nuclear Regulatory Commission has not provided criteria for an acceptable probability value for the SSE, obviously Intensity IX for Site A would be a reasonable estimate for firm site conditions. The best estimate for the probability of experiencing this environment is 0.4% for a 50-year service life. This assumes a maximum earthquake of $m_b = 7.5$.

The seismic risk analysis is subject to errors resulting from possible variations in intensity attenuation rates. The statistical variation in the attenuation curves could be considered in the probability calculation if a larger sample of records were available. It is difficult to estimate the effect of this variable, but percentages would probably be increased by a factor of about 1.05 to 1.10. There are also possible errors in the computed average intensity curves which are roughly estimated to be

about $\pm 10\%$ (i.e., a factor of 0.9 to 1.10). A more detailed study is required to more precisely assess the importance of these sources of error.

1.3.8 CONVERSION OF INTENSITY TO PEAK GROUND ACCELERATION

The problem of converting Mercalli Intensity to peak ground acceleration was discussed in Section 1.2.8. Here, the seismic risk has been left in terms of Mercalli Intensity. If a conversion is made to peak ground acceleration, the possible variations in the conversion relationship should be considered probabilistically. At present, this is not being done.

1.4 CONCLUSIONS

The results of this study indicate that there are three major problems in the practical application of seismic hazard analysis procedures:

1. For most regions in the United States, the historical record of seismic events is short and generally incomplete for small magnitude events. Because of the short duration of the record, there are inadequate data on large magnitude events. Extreme care must be exercised in the development of the relationship that defines the frequency of recurrence of earthquakes of different magnitude. Use of the historical record with no adjustment will usually underestimate the frequency of recurrence of small magnitude earthquakes and overestimate the frequency of recurrence of large magnitude events.
2. An average rate of attenuation of earthquake intensity with distance from the epicenter is normally used in seismic hazard assessment studies. It

is recommended that this variable be treated probabilistically. Additional study of this variable is needed.

3. Most of the information available for computing the earthquake intensity attenuation relationship for seismic hazard assessment studies is expressed in terms of either Rossi-Forel or Mercalli Intensities, and not in terms of peak ground acceleration. Also, the conversion of Mercalli

Intensity to peak ground acceleration is a source of significant error. Therefore, the results of seismic hazard assessment can be more reliably expressed in terms of Mercalli Intensity. If expressed in terms of peak ground acceleration, the potential error in the conversion from intensity to peak ground acceleration should be treated probabilistically in the seismic risk analysis. At present, this is not being done.

CHAPTER 2

STATISTICAL AND PROBABILISTIC CONSIDERATIONS IN HAZARD ANALYSIS

2.1 INTRODUCTION

Seismic hazard analysis techniques for representing input motions have a number of advantages over deterministic representations. First, they directly consider uncertainties associated with earthquake activity levels and can eventually be extended to include the additional uncertainties associated with intensity-attenuation rates, local site conditions, etc. In addition, information derived from seismic hazard analyses can be used in conjunction with cost vs. risk optimization studies and regulatory agency requirements to define seismic input criteria consistent with cost constraints and acceptable risk levels for the plant.

Estimates of recurrence rates of earthquake events are an important part of seismic hazard analyses for nuclear power plant sites. Formulae for recurrence estimates involve extreme value theory and Poisson stochastic processes. Probabilistic procedures can then be applied to the resulting data in order to perform either of the two types of seismic hazard analyses: the uniform seismicity approach or the variable seismicity approach. Problems associated with the practical application of seismic hazard analysis procedures were considered in Chapter 1 above.

2.2 RETURN PERIODS OF EARTHQUAKE EVENTS

2.2.1 RECURRENCE FORMULAE

The most commonly used approach for estimating the return period of earthquake events was originally proposed by Gutenberg and Richter (1954). In this procedure, curves relating earthquake magnitude or epicentral intensity are plotted vs. the frequency of

occurrence (or inverse of the return period). If this is done separately for different areas and the results normalized to account for differences in the sizes of these areas, the levels of seismic activity in each of the areas can be compared. This technique has the advantage of averaging many events and reducing the effects of isolated nontypical events; however, if the resulting curves are extrapolated to longer time intervals and to larger magnitude earthquakes than represented in the sample period, a number of important and debatable assumptions are involved (Allen et al., 1965).

The original development of earthquake recurrence formulae by Gutenberg and Richter was based on the use of earthquake magnitudes, although most of the available earthquake data is in terms of epicentral intensities. The general concepts of the use of recurrence formulae are the same regardless of whether magnitudes or epicentral intensities are used to define the earthquake events; however, there are some differences in magnitude-recurrence and epicentral intensity-recurrence formulae.

2.2.1.1 Magnitude-Recurrence Formula

The magnitude of an earthquake characterizes the size of the event in terms of energy radiated from its source and is essentially independent of the point of observation. This quantity is typically measured as either a local magnitude, based on the peak response of a defined instrument; a body wave magnitude, based on the response amplitude of a particular body wave type; or a surface wave magnitude, based on the response amplitude of long period seismic waves. Most of the reported magnitudes are determined on the basis of the local or body wave measures, although there are formulae

that interrelate the three magnitudes. Each of these magnitudes is based on a well-calibrated instrument, some knowledge of the material characteristics of the seismic wave travel path, and local conditions at the instrument station.

The form of the recurrence formula in terms of earthquake magnitude is

$$\log_{10} N = a - bM \quad (2-1)$$

in which N refers to the frequency of occurrence (which is the inverse of the return period), M refers to the earthquake magnitude, and a and b are constants determined from a tabulation and statistical assessment of the earthquake data. Typically, Equation 2-1 is plotted as a straight line on semilog paper, and this plot is termed the recurrence curve for the region under investigation.

Equation 2-1 represents the most widely used form for representing the frequency distribution of earthquake events in a region, and some studies of local seismicity have furnished excellent theoretical and empirical substantiation of this approach (Shakal and Willis, 1972). However, there are shortcomings associated with the application of Equation 2-1. For example, theoretical considerations of the energy released per unit time by very large earthquakes indicate that Equation 2-1 may not be applicable for such large events (Esteva, 1968). In addition, this equation presents problems with respect to theoretical evaluations of the slip rate per unit time associated with very small earthquakes (Rosenblueth, 1969). As a result, more complex recurrence rate formulae have been developed to provide an improved fit over a wider range of magnitudes. Newmark and Rosenblueth (1971) suggest that such formulae warrant consideration for special structures that must be designed to resist earthquakes with very long return periods. An example of a more complex approach to this problem is the

third asymptotic distribution associated with the theory of extreme values (see Sec. 2.2.2).

Errors introduced by Equation 2-1 are generally insignificant for the design of most conventional structures encountered in practice. For these structures, events with only moderate return periods are generally considered as the design basis, and data to fit Equation 2-1 are usually plentiful. However, for special structures, more difficulties are involved. Data regarding the availability of earthquake events with very long return periods, which must be considered in the design of these special structures, are typically quite sparse. Equation 2-1 provides conservative assessments of long return period events (see Sec. 2.2.2) although this degree of conservatism is uncertain. More refined procedures (see Newmark and Rosenblueth, 1971) might be appropriate for this case although results from such procedures are only as good as the available data on which to base the statistical model. Therefore, long return period events for the design of nuclear plants and other special structures must be defined using considerable care and judgment, with due regard for data uncertainties, cost constraints, and acceptable risk considerations. Procedures for testing the stability of recurrence curves can be used to test the adequacy of the data and improve the relationship (Stepp, 1972).

2.2.1.2 Interpretation of Results from Magnitude-Recurrence Formula

When evaluating earthquake recurrence data provided in the form of Equation 2-1, there are certain considerations associated with the interpretation of the data, including the definition of recurrence rate, the sizes of the areas being considered, and the time interval over which the data has been obtained.

When using Equation 2-1, not all seismologists have interpreted the recurrence rate parameter, N , in a consistent manner. Richter

(1958) used Equation 2-1 as a cumulative recurrence formula in which N is the number of events per year of magnitude M or greater; this interpretation was used in a number of seismic risk assessments by Vanmarcke and Cornell (1972). Other seismologists have interpreted N as the number of events per year within a magnitude interval of $1/2$ unit, where M is the magnitude at the center of the interval (Housner, 1970a; Hileman et al., 1973). Therefore, it is necessary to compare the definitions of the recurrence rate when comparing recurrence curves from different sources.

Another factor to consider when interpreting recurrence curves is the sizes of the various regions being investigated. If the absolute size of a region is excessively large, its recurrence curve will represent an average seismicity for the entire region that may not be representative of a particular site; this is especially important if the overall region contains areas of markedly higher or lower seismicity. If the region is too small, the resulting recurrence curve may ignore earthquake events centered outside of the region that could conceivably influence the seismic risk within the region. Also, the data are usually not sufficient to provide a stable relationship.

The total time period over which the seismic data is obtained also has an important influence on the interpretation of the resulting recurrence curves. The extrapolation of recurrence curves to longer return periods and to larger magnitudes than considered in the data may lead to important misinterpretations of the seismicity of the area. The recurrence curve concept is valid only for those magnitude levels for which a sufficient number of events have occurred within the total time period of the data. For this reason, recurrence curves may not always provide an adequate indication of the recurrence of large magnitude earthquakes with long return periods, since the total time over

which the data was obtained is often insufficient to permit an adequate statistical sampling of these large earthquake events.

2.2.1.3 Magnitude-Recurrence Rates for Southern California

A study by Hileman et al. (1973) contains a systematic listing of over 15,000 earthquake events recorded throughout Southern California in the 41-year period from 1932 through 1972 by the Caltech Seismological Laboratory. This data has been used to develop recurrence curves using Equation 2-1 for various regions within Southern California. Figure 2-1 contains a comparison of a number of these regional recurrence curves with the curve for the entire Southern California area and illustrates a number of interesting trends:

- a. The recurrence curves of various regions in Figure 2-1 are considerably different from each other and from the combined recurrence curve of Southern California as a whole.
- b. Recurrence curves can be very sensitive to the time span of the data, especially if this time span is small. This is illustrated by the substantial differences between the recurrence curve for the White Wolf fault zone for the period from 1932-1951 relative to the curve for the period from 1952-1971. It is noted that the latter curve includes the 1952 Kern County earthquake.
- c. The recurrence curve for the Los Angeles area is roughly comparable to the average curve for the entire Southern California region. The regions with the greatest recurrence curves for the time interval considered are the White Wolf fault zone (1952-1971) and the Imperial Valley.

(a) Recurrence curve comparisons

Legend	Location and Time Span of Data	Area (km ²)	Number of Events	Minimum Magnitude	Slope of Recurrence Curve, b(1)
————	Southern California 1932-1971)	238,600	3,404	3.5	0.98
-----	Los Angeles Area (1932-1971)	26,622	1,373	3.0	0.93
-----	White Wolf Fault Area (1932-1951)	8,400	93	3.0	0.85
-----	White Wolf Fault Area (1952-1971) (2)	8,400	986	3.0	0.87
-----	Parkfield Area (1932-1971)	15,000	694	3.0	0.80
-----	Imperial Valley Region (1932-1971)	15,102	786	3.5	0.85
.....	Santa Barbara Channel (1932-1971)	10,200	328	3.0	1.00
●-----	Northern Sierra Nevada (1932-1971)	10,600	144	4.0	1.01
○-----	Southern Sierra Nevada (1932-1971)	8,450	555	3.0	0.98

Notes:

- (1) The parameter b corresponds to the slope of the recurrence curve $\log_{10} N = a - bM$
- (2) The seismicity data for the White Wolf fault zone for the time period 1952-1971 includes the 1952 Kern County earthquake.

(b) Descriptions of regions investigated

FIGURE 2-1. RECURRENCE CURVES FOR SOUTHERN CALIFORNIA REGIONS (Hileman et al., 1973)

The lowest recurrence curves result from earthquake data from the White Wolf fault zone (prior to 1952) and the Santa Barbara Channel.

2.2.1.4 Epicentral Intensity-Recurrence Formula

The intensity of an earthquake is a subjective measure of its effect on structures, people, and the terrain in the vicinity of the event. For present-day earthquakes, intensity distributions are typically obtained through interviews with people who experienced the earthquake as well as from observations of damage in affected areas. Intensities from historical earthquake events have been obtained by reviewing newspapers, journals, books, or other possible sources of information. The areas nearest the source of the earthquake generally incur the greatest effects and therefore are assigned the highest intensity (termed epicentral intensity), although this depends on a number of factors such as the type of soil, the type of construction near the epicenter, etc. As the distance from the earthquake source increases, the earthquake effects and resulting intensities tend to decrease in a manner dependent on the tectonics and geologic characteristics of the area, reflecting a greater transmissibility of waves in one direction or another or, to a greater extent, the manner in which the earthquake originates. The 12-step Modified Mercalli Intensity scale is now most often used in the United States although other intensity scales have been used in the United States and in other parts of the world.

Epicentral intensities have been used as a basis for defining return periods in regions of moderate-to-low seismicity where very large time intervals must be considered in order to develop an adequate sampling of past earthquake events. The rationale for using intensities in this manner arises from the fact that

they provide the only means for interpreting historical data or for evaluating the effects of earthquakes in regions where only limited magnitude measurements have been obtained. However, the interpretation of intensity data over a long time period with regard to recurrence rate definition may involve some difficulty. For example, since construction practices in most areas of the world have improved substantially with time, the use of structure damage reports does not always provide a consistent basis from which to estimate intensities when applied to earthquake events occurring many years apart. Furthermore, many areas have also experienced considerable changes in population density over the past few hundred years; therefore, intensities from earthquakes felt long ago in regions that were sparsely populated at the time are not as reliable as intensities from more recent earthquakes occurring in these regions after they have become more heavily populated. As a result, the interpretation of return periods using intensity data from historic earthquakes along with more recent events must be carried out with considerable caution.

The following interrelationship between recurrence rate N and epicentral intensity I_0 is used:

$$\log_{10} N = a' - b' I_0 \quad (2-2)$$

where a' and b' are constants that are determined from a compilation of the epicentral intensity data for the region. It is of interest to note that Equation 2-2, which expresses recurrence rates in terms of epicentral intensities, has the same form as Equation 2-1, which considers the magnitude as a basis for estimating recurrence rates. In fact it is possible to interrelate Equations 2-1 and 2-2 if a direct proportionality between magnitude and epicentral intensity can be established. One region within the continental United States for which

this type of proportionality has been established is California where, for moderate-to-small earthquakes and firm soil conditions, Gutenberg and Richter (1956) developed the following magnitude-epicentral intensity relationship

$$M = 1 + \frac{2I_0}{3} \quad (2-3)$$

Substituting Equation 2-3 into Equation 2-1 yields Equation 2-2 with the following interrelationships between the intensity-recurrence rate constants (a' and b') and the magnitude-recurrence rate constants (a and b):*

$$a' = a - b$$

$$b' = 2b/3$$

(2-4)

There are numerous examples of epicentral intensities used as a basis for defining return periods. For example, Bollinger (1973) utilized intensity data to establish that the recurrence rate for the southeastern United States could be defined by Equation 2-2 with $a' = 3.01$ and $b' = 0.59$. The area considered and the resulting recurrence curve are given in Figure 2-2. Fischer and McWhorter (1972) developed the

FIGURE 2-2. RECURRENCE CURVE FOR SOUTHEASTERN UNITED STATES (Bollinger, 1973)

*Equations 2-3 and 2-4 are strictly valid only for California events, since "...in other regions, the prevailing hypocentral depth and differences in crustal structure may result in a different relationship between magnitude and intensity..." (Richter, 1958). However, these relationships have been used in other areas where the tectonics are similar to California (Couch and Deacon, 1972).

intensity recurrence curves shown in Figure 2-3 when performing a seismic microregionalization investigation of New York state.

The previous discussion on the sensitivity of results to the size of the area and to the total time period of the data can be applied to

(a) Microzonation areas and past earthquake events

(b) Recurrence curves

FIGURE 2-3. SEISMIC REGIONALIZATION OF NEW YORK STATE (Fischer and McWhorter, 1972)

epicentral intensity-recurrence curves as well. The interpretation of return periods using epicentral intensity data must carefully consider the absolute size of the area, differences in the sizes of two areas whose recurrence rates are being compared, and the total time period of the data relative to the longest return period of interest in the investigation.

2.2.2 EXTREME VALUE THEORY

2.2.2.1 Summary of Theory

The theory of extreme values is based on the assumptions that the prevailing conditions will be valid in the future and that observed largest values are independent of each other. From this, the return period, $T(M)$, defined as the mean time required for an earthquake of magnitude M to occur, is

$$T(M) = \frac{1}{1 - \phi_n(M)} \quad (2-5)$$

where $\phi_n(M)$ is the probability that a given magnitude M will be the largest among n independent earthquake events all having the same probability function.

Since the probability function for $\phi_n(M)$ is not known, asymptotic forms of distributions must be used. These forms require the specification of certain parameters which can be estimated from the earthquake data of the region under investigation (Gumbel, 1960). Three asymptotic distributions of extremes have been developed. Only the first and third distributions are considered applicable for the evaluation of maximum magnitude earthquakes (Yegulalp and Kuo, 1974) and are expressed as follows:

First Distribution:

$$\phi_n^{(1)}(M) = \exp \left\{ -\exp [-\alpha_n (M - u_n)] \right\} \quad (2-6a)$$

where

α_n = Extremal intensity function > 0

u_n = Characteristic largest value

$$\phi_n^{(1)}(u_n) = e^{-1}$$

Third Distribution:

$$\phi_n^{(3)}(M) = \exp \left\{ - \left[\frac{w - M}{w - u_n} \right]^{k_n} \right\} \quad (2-6b)$$

where

k_n = Shape parameter > 0

w = Upper limit of largest values $\geq M$

u_n = Characteristic largest value $< w$

$$\phi_n^{(3)}(u_n) = e^{-1}; \quad \phi_n^{(3)}(w) = 1$$

Note that the first asymptotic distribution is essentially equivalent to the magnitude-recurrence formula (Eq. 2-1); both involve two arbitrary parameters with which to fit the data and, in fact, Equation 2-1 can be derived directly from Equation 2-6a. The third asymptotic distribution involves an additional arbitrary parameter that can be adjusted to provide an improved representation of the data over a wider range of events. Furthermore, the first asymptotic distribution presumes neither an upper bound nor a lower bound to the magnitude M ; in contrast to this, the third distribution considers an upper limit to M . Since there is probably a maximum possible magnitude at any given location, as determined by the strength and thickness of its rock crust, the third asymptotic distribution would seem to provide a more physically meaningful representation of maximum magnitudes than the first distribution. However, whether this truly represents an advantage when predicting return periods of maximum magnitude events depends on the availability of suitable data.

2.2.2.2 Applications of Extreme Value Theory

One of the objectives in presenting examples of extreme value theory is to demonstrate that when applied to large regions, the data show asymptotic maximum magnitude earthquakes that vary from one area to another. The first demonstrations of the applicability of extreme value theory to the estimation of the probable occurrence of maximum magnitude earthquakes were carried out by Nordquist (1945). In his analysis of world data from 1904-1939, as well as Southern California data, Nordquist showed that these probable occurrences correlate reasonably well with the first asymptotic distribution of largest values (Eq. 2-6a), which plots as a straight line on Gumbel probability paper.

Since the initial work by Nordquist, the theory of extreme values has been used to analyze earthquakes in various regions of the world such as New Zealand (Dick, 1965), Canada (Milne and Davenport, 1969), Europe (Karnik and Hubernova, 1968; Schenkova and Karnik, 1970), and the North Circum-Pacific area (Shakal and Willis, 1972). Each of these studies found the theory of extreme values to fit the distribution of largest earthquakes fairly well.

Yegulalp and Kuo (1974) applied the theory of extreme values to 46 regions of the world. The objectives of this study were to (1) assess the applicability of results obtained using the first asymptotic distribution (FAD) and the third asymptotic distribution (TAD) and (2) utilize results from extreme value theory to develop recurrence curves for each region and estimate return periods of future events. Data for each region were originally developed by Gutenberg and Richter (1954) for the period 1904-1952 and by Rothe (1972) for the period 1953-1965. The regions considered in the study are listed in Table 2-1.

To fulfill the first objective, Yegulalp and Kuo utilized least-squares techniques to provide the best possible fits of the FAD and TAD to the 1904-1965 earthquake data for the various regions of the world. Next, to meet the second objective, the TAD results based on this same data were used to develop and compare recurrence curves for the various regions. In addition, once the predictability and stability of these statistical results were established,* return periods of future maximum magnitude earthquakes were estimated for each region.

Results of the comparisons of FAD and TAD recurrence curves with 1904-1965 data are shown in Figure 2-4. This figure indicates that for some regions the FAD and TAD predictions compare closely with each other and with 1904-1965 data. However, for other regions, this comparison indicates differences that follow certain trends:

- a. Where differences exist, the TAD predictions generally compare closer with 1904-1965 data than do the FAD predictions. This trend can be attributed to the fact that the TAD involves one additional parameter for fitting the data.

*The predictability of the results was evaluated by using the 1904-1952 data of Gutenberg and Richter to predict the occurrence of maximum magnitude earthquakes during 1953-1965; these predictions were then evaluated by comparisons with the actual data for 1953-1965 developed by Rothe. Good comparisons between predictions and the Rothe data were obtained. The stability of the data was tested by comparing TAD parameters from 1904-1952 with those parameters based on 1904-1965 data. These various sets of parameters compared closely. On the basis of these two series of tests, Yegulalp and Kuo concluded that the results were both predictable and statistically stable. However, for long return period events, this conclusion is debatable, as discussed subsequently.

TABLE 2-1. TABULATED RESULTS FROM EXTREME VALUE STUDY OF EARTHQUAKE RETURN PERIODS (Yegulalp and Kuo, 1974)

Region No.	Name of the Region	N Total Sample Size (1904-1965 data)	Return Period, years		
			20	50	100
1	Aleutian Islands, West Alaska	62	8.3	8.7	8.9
2	East Alaska, British Columbia	58	7.5	7.8	8.0
3	California, Nevada	60	7.6	8.1	8.5
4	Gulf of Mexico	59	7.1	7.2	7.2
5	Mexico, south of 28°	59	8.3	8.6	8.8
6	Central America	62	7.8	8.0	8.2
7	Caribbean	60	7.6	7.9	8.1
8	South America, north of 37°	60	8.5	8.8	9.1
9	South America, south of 37°	47	7.7	8.0	8.2
10	South Antilles	56	7.6	7.8	7.9
11	Macquaire, New Zealand	52	7.9	8.2	8.4
12	Kermadec and Tonga Islands	59	8.2	8.5	8.7
13	Fiji	43	7.0	7.2	7.2
14	New Hebrides	57	8.0	8.4	8.6
15	Solomon Islands to New Britain	56	8.0	8.2	8.3
16	New Guinea	60	8.0	8.3	8.5
17	Caroline Islands	57	7.5	7.9	8.2
18	Marianne Islands	61	7.9	8.3	8.6
19	Japan-Kamchatka	62	8.5	8.9	9.1
20	Ryukyu Islands	62	8.1	8.6	8.8
21	Formosa	57	7.9	8.1	8.3
22	Philippine Islands	59	8.2	8.4	8.6
23	Celebes	61	8.0	8.4	8.7
24	Sunda Islands	57	7.9	8.1	8.2
25	Andaman Islands to Burma	48	7.5	7.6	7.7
26	Szechuan, Southern Tibet	60	7.9	8.3	8.5
27	Kansu to Sinkiang	56	7.4	7.7	7.8
28	Mongolia	52	7.5	8.0	8.2
29	Iran-Urals	57	7.4	7.5	7.6
30	Asia Minor-Levant-Balkans	62	7.7	8.0	8.2
31	Western Mediterranean	58	7.1	7.6	7.8
32	Atlantic Ocean	54	7.3	7.5	7.6
33	Indian Ocean	62	7.4	7.6	7.7
34	North America	40	7.0	7.4	7.6
35	Brazilian Shield	*	---	---	---
36	Central and Western Europe	*	---	---	---
37	Africa	56	7.0	7.1	7.3
38	Australia	51	6.3	6.7	7.0
39	Pacific Basin	37	6.8	7.0	7.1
40	Arctic Belt	58	6.8	7.0	7.2
41	Eastern Siberia	55	6.7	7.0	7.1
42	Baffin Bay to Bering Sea	46	6.9	7.3	7.4
43	Southeastern Pacific	54	7.0	7.1	7.1
44	Eastern Pacific	40	6.4	6.5	6.5
45	Indian Antarctic Swell	45	6.9	7.0	7.1
46	Manchuria to Sea of Okhotsk	61	7.5	7.7	7.8
47	Balucistan	*	---	---	---
48	Hindu Kush and Pamir	59	7.9	8.2	8.3
49	Eurasian Stable Mass	*	---	---	---
50	Antarctic	*	---	---	---
51	Rumania	58	6.5	7.2	7.7

* Rejected due to inadequate sample size.

FIGURE 2-4. COMPARISONS OF APPLICATIONS USING FIRST AND THIRD ASYMPTOTIC DISTRIBUTIONS (Yegulalp and Kuo, 1974)

- b. The FAD always predicts shorter return periods for larger magnitude earthquakes; in some cases, differences between FAD and TAD predictions for these large events are substantial. This follows from the lack of an upper bound to the earthquake magnitude in the FAD, where, for the TAD, an upper bound estimate is required.
- c. For moderate earthquakes, differences between the TAD and FAD return

period predictions are generally small. Where there are differences, the FAD generally results in slightly longer return period estimates than the TAD. Again, this may follow from the fact that the maximum earthquake magnitude is bounded for the TAD and is unbounded for the FAD.

These trends regarding the predicted occurrence of events with long return periods have particular importance for nuclear plants because of their stringent safety requirements.

As noted previously, the FAD corresponds to the magnitude-recurrence formula given in Equation 2-1, whereas the TAD represents an apparent refinement to this equation. However, any apparent advantages gained through this increased refinement of the TAD must be interpreted with regard to the amount of reliable data available to develop the statistical model. For example, Yegulalp and Kuo used 61 years of data to estimate 150-year earthquakes, and this may not provide an adequate statistical sample of these large events. An analogous problem invariably arises at nuclear plant sites where there is seldom more than 100 years of reliable data that must be used to estimate the occurrence of earthquake events with considerably longer return periods. This suggests that, when using extreme value theory to estimate earthquake recurrence rates for a particular seismic region, the availability of suitable earthquake data should influence the selection of the FAD vs. the TAD, as well as the interpretation of results from the subsequent analysis. The more refined TAD will provide an improved fit over a wider range of earthquake magnitude levels, although it will not substitute for an inadequate sample size, as often exists for long return period events. The simpler FAD has the advantage of being conservative for long return periods.

In addition to assessing the relative reliability of the FAD and TAD, Yegulalp and Kuo also developed recurrence curves for the various regions using the TAD fitted to the 1904-1965 data. Figure 2-5 indicates that these regional recurrence curves are upper bounded by the asymptotic distribution curve of the world and are lower bounded by the curve of Region 38, Australia. As expected, the circum-Pacific belt exhibits the highest curves, followed by the Alpide Belt and finally by the oceans and stable continental masses.

As a third application of the extreme value theory, Yegulalp and Kuo used the 1904-1965 data to estimate future maximum magnitude earthquakes corresponding to return periods of 20, 50, and 100 years for the 46 regions considered in the study; these predicted return periods are shown in Table 2-1. The predictions indicate that the Alaska and California-Nevada regions may expect 50-year maximum magnitude earthquakes of 8.7 and 8.1 respectively, while for the remainder of North America, a 50-year earthquake with a maximum magnitude of 7.4 is predicted. As previously noted, the data base from which these earthquake levels were estimated is limited, although comparisons of the relative magnitudes of the 20-, 50-, and 100-year events for various regions provide a measure of their relative seismicities.

2.2.3 POISSON PROCESSES IN EARTHQUAKE FORECASTING

2.2.3.1 Description of Poisson Processes

The foregoing procedures are applied to one type of earthquake forecasting -- namely, the average time interval between events of a given size (i.e., the return period). A slightly different forecasting approach uses probabilities of occurrence of given sized earthquakes within a given time interval. This approach has been widely used in the earthquake engineering field (Newmark and Rosenblueth, 1971; Benjamin, 1968).

Because of uncertainties regarding the underlying processes that cause earthquakes, earthquake events must be regarded as being random in nature. As a result, they are often forecast using Poisson stochastic processes, a well-known approach for predicting the occurrence of random events. The basic assumptions of this approach, as applied to the earthquake problem, are:

(a) Northern and western circum-Pacific belt

(b) Eastern circum-Pacific belt

(c) Alpine belt and adjacent regions

(d) Stable continental masses and oceans

NOTE: SEE TABLE 2-1 FOR NUMBERED REGION LOCATIONS

FIGURE 2-5. USE OF EXTREME VALUE THEORY TO ESTIMATE EARTHQUAKE RECURRENCE CURVES FOR VARIOUS REGIONS OF WORLD (Yegulalp and Kuo, 1974)

- a. Stationarity. The probability of an earthquake of epicentral intensity I_0 , during the small time interval from t to $t + \Delta t$, is approximately $\mu(I_0) \Delta t$ for any t , where $\mu(I_0)$ is the occurrence rate of events of epicentral intensity I_0 and is time independent.
- b. Time Independence. An earthquake event of epicentral intensity I_0 is equally likely to be found in any unit-time interval along the time axis. Furthermore, the number of such events in any interval of time is independent of the number in any other non-overlapping interval of time.
- c. Spatial Independence. Within a given time interval, an earthquake event of epicentral intensity I_0 is equally likely to be centered anywhere in the region of interest.

From these assumptions, it can be shown (Benjamin and Cornell, 1970) that the probability of occurrence of r earthquake events of epicentral intensity I_0 within a time span t and with a mean occurrence rate $\mu(I_0)$ is

$$p(r, t) = \frac{1}{r!} \exp\{-\mu(I_0) t\} \cdot [\mu(I_0) t]^r \quad (2-7)$$

where $\mu(I_0)$ for each epicentral intensity is typically defined from the past earthquake data for the region. For example, if historical records for a time period of 100 years indicate the occurrence of 5 earthquakes of epicentral intensity VII, the mean occurrence rate for events of this size is 5/100, or 0.05. Then the probability of at least one such event in, say, the next 5 years is computed as

$$\begin{aligned} P_p &= 1 - p(0, t) \\ &= 1 - \frac{1}{0!} \left\{ \exp[-0.05 \cdot 5] \right\} \cdot [0.05 \cdot 5]^0 \\ &= 1 - 0.779 = 0.221 \end{aligned}$$

In practice, computations of this type are repeated for other epicentral intensities and time intervals, from which curves relating probability to epicentral intensity for different time intervals can be obtained. To use these curves, the engineer first selects a particular probability vs. intensity curve corresponding to a time interval consistent with the planned life of the structure. The probability of occurrence can then be used as a basis for selecting a design earthquake event that represents an acceptable level of risk for the structure. For example, a design earthquake event for a nuclear plant should be based on a lower probability of being exceeded than, say, an office building or warehouse which has no immediate function after an earthquake except to protect its inhabitants until they can be evacuated.

As a second application of this approach, consider the problem of determining equivalent fatigue cycles for the design of nuclear plant internal elements. These equivalent cycles are typically based on multiple applications of different sized earthquakes; the number of these applications is based on earthquake recurrence curves and on the planned life of the plant. It is possible to select the number of applications of each sized earthquake in terms of probabilities of occurrence, rather than solely on recurrence rates. To illustrate, consider the example where examination of recurrence curves for fatigue design purposes suggests three applications of an epicentral intensity VI event during the 50-year life of the plant. In order to evaluate this assumption, it is necessary to utilize Equation 2-7 to compute the probability that more than three events of epicentral intensity VI will occur during a 50-year period. This is computed as

$$\begin{aligned} p(r > 3, 50) &= 1 - p(0, 50) - p(1, 50) \\ &\quad - p(2, 50) - p(3, 50) \end{aligned}$$

where each term on the right hand side is computed from separate applications of Equation 2-7. If this computed probability falls within an acceptable risk level, then the assumed number of occurrences of epicentral intensity VI events is safe. Otherwise, the number of occurrences must be increased, in order to reduce the probability of being exceeded so that it falls within the specified risk level. It is clear that this approach can be applied to other events of other epicentral intensity levels to determine if the assumed number of applications of these events is also safe. The assumed number of occurrences of each sized earthquake would therefore be based on a common acceptable risk level; in this manner a consistent basis for fatigue design is established.

The primary advantage of the Poisson process is its simplicity and wide prior applicability to the earthquake problem (e.g., in Rosenblueth, 1964, and Benjamin, 1968). However, the approach has certain shortcomings. For example, experience with foreshocks and aftershocks does not agree with the assumptions inherent in the Poisson law. Furthermore, even if one disregards foreshocks and aftershocks, evaluations of past data have indicated that earthquakes generally tend to cluster in time and space, rather than being time- and space-independent as assumed for Equation 2-7. Finally, the available earthquake data in a region may not be of sufficient length to define the mean rate of occurrence of earthquake events (i.e., the parameter μ (I_0)); this is particularly important for large earthquake events of the type considered in nuclear plant design applications and for which existing data is often limited. Of course, all procedures for estimating return periods and for earthquake forecasting will have certain problems when only limited data is available.

2.2.3.2 Bayesian-Poisson Approach

To reduce uncertainties associated with estimating the mean occurrence rate, Bayesian statistics can be used in conjunction with Poisson processes to obtain improved earthquake forecasting capabilities. Bayesian statistics treat unknown parameters as random variables, thereby recognizing the uncertainties associated with the true values of these parameters, as well as inherent uncertainties associated with the underlying distribution of the variable. For example, let X represent the occurrence of an earthquake with a given epicentral intensity, and let μ represent the occurrence rate, which is often known only from a limited amount of available data. Then, Bayesian concepts can be used to define a new probability distribution

$$f_X(x) = \int f_X(x|\mu) f_\mu(\mu) d\mu \quad (2-8)$$

where $f_\mu(\mu)$ is the probability distribution of μ and $f_X(x|\mu)$ is the conditional probability of x given μ . Equation 2-8 can be interpreted as a weighted average of all possible distributions $f_X(x|\mu)$ which are associated with different values of μ (Benjamin and Cornell, 1970). Hence, the unknown parameter μ has been "integrated out" of the equation and will not appear in $f_X(x)$. Furthermore, as more data become available, the distribution $f_X(x)$ will approach the true distribution of X , since the distribution of μ will become more centralized about the true value of the parameter.

Another important advantage of Bayesian statistics is that uncertainties associated with the mean occurrence rate are conservatively included in the estimated probability of occurrence for the random variable X (Donovan, 1974). This follows from the fact that the probability distribution $f_X(x)$ as computed from Equation 2-8 will be wider (i.e., have a larger variance) than the true distribution of X , since

the former incorporates both inherent and statistical uncertainties (Benjamin and Cornell, 1970).

Bayesian-Poisson occurrence forecasting techniques were applied to earthquakes by Benjamin (1968). These techniques were used to develop the following expression for computing the probability of observing r_o future Poisson events in time t_o , having observed r events in time t :

$$P [r_o, t_o; r, t] = \frac{(r + r_o)!}{r! r_o!} \frac{\left(\frac{t_o}{t}\right)^{r_o}}{\left(1 + \frac{t_o}{t}\right)^{r_o + r + 1}} \quad (2-9a)$$

Hasofer (1969) has noted that Equation 2-9a is based on an assumed prior distribution of the mean rate of occurrence μ that is uniform over the range $0 \leq \mu \leq \infty$. To circumvent certain difficulties associated with this assumption, Hasofer assumed that μ could be reported by a gamma density function, which results in the following expression for estimating probabilities of earthquake events:

$$P' (r_o, t_o; r, t) = \frac{(r + r_o - 1)!}{r_o! (r - 1)!} \frac{t_o^{r_o} t^r}{(t + t_o)^{r + r_o}} \quad (2-9b)$$

Equations 2-9a and 2-9b predict similar probabilities when the amount of information is large (r, t are large when compared to r_o, t_o). When little information is available, the expressions yield different results. Conceivably other prior distributions for μ could also be assumed which, in turn, would result in still other estimates of earthquake probabilities, particularly when the amount of information is small. This indicates uncertainties associated with the predictions of future events based on limited data. Furthermore, in view of the assumptions involved in the use of Poisson occurrence and

subjective intensity estimates to define earthquake events, it is difficult to justify the use of one assumed distribution of μ instead of another (Benjamin, 1969). In this spirit, Equation 2-9a will be used here, although other earthquake probability expressions could be derived, corresponding to different assumed distributions of μ .

To illustrate the application of Equation 2-9a, consider the previous conditions analyzed using Poisson forecasting techniques. In this, 5 events of epicentral intensity VII were observed over a 100-year period and it is desired to estimate the probability of occurrence of at least one such event in the next 5 years. This is computed as

$$\begin{aligned} P_{BP} &= 1 - P [0, 5; 5, 100] \\ &= 1 - \frac{(0 + 5)!}{0! 5!} \frac{\left(\frac{5}{100}\right)^0}{\left(1 + \frac{5}{100}\right)^{0+5+1}} \\ &= 1 - \frac{1}{(1.05)^6} = 0.254 \end{aligned}$$

These computations can be used to define curves relating probability of occurrence to epicentral intensity for different time intervals, and can, in turn, serve as a basis for selecting a design earthquake event consistent with an acceptable level of risk for the structure. In addition, Bayesian-Poisson approaches can also be used to define multiple occurrences of smaller events for fatigue design purposes.

Comparisons of the simple Poisson approach (Eq. 2-7) and the Bayesian-Poisson method (Eq. 2-9a) given in Table 2-2 indicate that the Bayesian-Poisson approach has the advantage of providing conservative probability estimates when the time length of the data is short relative to the return period of interest. This case is of particular importance for nuclear plants which must be designed to resist earthquakes

TABLE 2-2. COMPARISONS OF POISSON AND BAYESIAN-POISSON FORECASTING PROCEDURES

Factor	Comparison
Large Quantity of Data Relative to Return Period of Interest	Basic Poisson approach (Eq 2-7) and Bayesian-Poisson approach (Eq 2-9) will converge to identical forecasts
Small Quantity of Data Relative to Return Period of Interest	Equation 2-9 will result in larger probabilities of occurrence for given return period than Equation 2-7, reflecting conservative nature of Bayesian approach. Of course, as previously noted, probabilities computed from small quantities of data will be sensitive to particular assumptions associated with distribution of occurrence rate, i.e., there are still important uncertainties associated with use of limited data, even when Bayesian techniques are applied
Prior Non-occurrence of Given Event	If, say, an earthquake of epicentral intensity VII had never been observed within region, Equation 2-7 would result in zero probability of occurrence of future events having this intensity, even though it might not be impossible for event of this intensity to occur. Equation 2-9 would result in nonzero probability of occurrence for this case
Influence of Increase in Amount of Data	Equation 2-9 is sensitive to influence of increased data whereas Equation 2-7 is not. For example, Equation 2-7 would yield same probability for 5-year return period regardless of whether 1 event had been observed in 10 years or 10 events had occurred in 100 years. Equation 2-9 would yield higher probability of occurrence for shorter record length and would converge toward Equation 2-7 results as record length increases

having very long return periods. As the amount of data increases, forecasts from the Bayesian-Poisson methods converge.

2.3 SEISMIC HAZARD ANALYSES

Seismic hazard analyses that provide probability levels and return periods associated with a given intensity of ground shaking require data that define the frequency of occurrence of earthquake events and the attenuation of ground shaking with distance from the fault. Probabilistic procedures are then applied to this data in order to perform the seismic hazard analyses. Two basic types of seismic hazard analyses are the uniform seismicity approach and the variable seismicity approach (see Chapter 1 above).

2.3.1 UNIFORM SEISMICITY APPROACH

The probabilistic method developed by Housner (1970b) has been applied to assess the

seismic hazard in California and later extended to the Puget Sound area by Ts'ao (1974). Its basic assumptions are that the specified region has uniform seismicity, and that any one earthquake occurrence within the specified region is independent of any other occurrence and is Poisson distributed.* Since the method disregards the possibility that all portions of the region may not have equal seismicity, the

*In the seismic risk analysis procedures presented here, the random variable corresponds to the site intensity I caused by all possible events in the region (contrast to the use of epicentral intensity I_e for the Poisson distribution, discussed previously). This variable is dependent not only on the arrival rate of past earthquake events, but on the intensity-attenuation rate assumed for the region as well. Except for these differences, the general assumptions and applicability of Poisson processes discussed earlier are also applicable to the seismic risk approaches described here.

resulting calculations will provide probabilities that are too high for some areas and too low for others. Therefore, this approach is most applicable to those regions that do not exhibit any strong localized patterns of higher or lower seismicity; fortunately many potential nuclear plant sites in the United States and around the world fall into this category.

Six steps involved in performing probability calculations using this approach are:

1. Develop a relationship between the earthquake recurrence rate and magnitude or epicentral intensity.
2. From isoseismal maps of past earthquakes in the region, develop intensity-attenuation curves corresponding to various epicentral intensity levels. A single set of these curves is considered to apply for the entire region.
3. Define an appropriate interrelationship between site intensity and peak acceleration.
4. Perform the probability calculations for the region. In this, it is assumed that any earthquake occurrence is independent of any other occurrence. From this, it can be shown that the probability of occurrence of a given site intensity, I , within L years, $P(I, L)$, is

$$P(I, L) = 1 - \exp \left[- \sum_{I_0} \frac{a(I_0, I) \cdot N(I_0) \cdot L}{A} \right] \quad (2-10)$$

where

A = Total area of region
 $a(I_0, I)$ = Area experiencing intensity I due to earthquake event with epicentral intensity I_0

(as determined in Step 2)

$N(I_0)$ = Mean number of occurrences of earthquakes with epicentral intensity I_0 per unit period of time (as determined in Step 1)

$\sum_{I_0}$ = Summation over the entire range of epicentral intensities of earthquake events centered within the region.

5. The peak acceleration corresponding to the intensity I in Equation 2-10 is readily obtained by utilization of the intensity-acceleration conversion relationships defined in Step 3 above.
6. The return period for this level of ground shaking can be computed as:

$$T(I) = \frac{1}{\sum_{I_0} \frac{a(I, I_0)}{A} \cdot N(I_0)} \quad (2-11a)$$

and is related to the probability, $P(I, L)$ through the expression:

$$P(I, L) = 1 - \exp \left[- \frac{L}{T(I)} \right] \quad (2-11b)$$

Results of the applications of this approach to California and the Puget Sound areas are shown in Figure 2-6 for a structure life of 50 years. The results of these applications, in terms of curves relating the probability of occurrence to the peak acceleration, clearly indicate the relative seismic hazard of the two areas investigated; as expected, California exhibits the greater seismic hazard.

FIGURE 2-6. EXAMPLE APPLICATIONS OF UNIFORM SEISMICITY APPROACH FOR PERFORMING SEISMIC HAZARD ANALYSES

2.3.2 VARIABLE SEISMICITY APPROACH

The primary feature of the variable seismicity approach is that variations in the seismicity of the region, due to predominant active faults for example, can in theory be considered (Cornell and Vanmarcke, 1969; Cornell, 1970). The method also accounts for different source geometries, and superimposes the effects of the earthquakes at each source to develop probability estimates for the ground motions at a particular location within the region being investigated. The approach is based on the assumption that occurrences in time of the i th potential event are random (i.e., Poisson arrivals) with a constant average rate per year. Furthermore, it is assumed that earthquake events that have actually occurred within a line or area source are equally likely to have occurred anywhere within that source.

The variable seismicity approach first requires that the engineer define each of the various potential earthquake sources; these could be line sources such as faults, point sources such as relatively short faults located

far from the site, or area sources such as tectonic provinces in regions where earthquakes cannot be definitely correlated with a causative fault. For each of these potential sources the engineer must then assign (1) an activity level in the form of an average annual rate of occurrence of earthquakes of magnitude M_0 or greater, and (2) an attenuation rate for ground motion as a function of magnitude and distance. These assigned activity levels and attenuation rates are presumably based on the seismic history and tectonics of this and other geologically similar regions.

The first computational step is to relate the ground motion Y (displacement, velocity, or acceleration) to the earthquake magnitude M and the focal distance R :

$$Y = b_1 \exp(b_2 M) R^{-b_3} \quad (2-12)$$

where b_1 , b_2 , and b_3 are selected from the geologic conditions of the area. Esteva (1970) showed that for the western United States $b_1 = 2000$, $b_2 = 0.8$, and $b_3 = 2$ for the case where Y is the peak acceleration (cm/sec^2), R

is in units of kilometers, and M is the Richter magnitude.

The next step is to express the frequency of occurrence of earthquakes at each source. The computational procedure assumes that the frequency distribution in terms of magnitude is

$$F(M) = \frac{1 - e^{-\beta(M - M_0)}}{1 - e^{-\beta(M_1 - M_0)}}; \quad M_0 \leq M \leq M_1 \quad (2-13)$$

where

M_0 = Minimum magnitude of interest

M_1 = Maximum possible magnitude

β = $b \cdot \ln 10$

b = Magnitude coefficient in the recurrence rate expression

$$\log_{10} N = a - bM$$

From Equations 2-9 and 2-10 and the assumptions indicated above, and upon combining results from all potential earthquake sources, the probability that the peak motion at the site exceeds some critical value $\tilde{Y}$ during a time period T is

$$P(Y > \tilde{Y}, T) = 1 - \exp \left\{ e^{\beta M_0} T vG \left[\frac{\tilde{Y}}{b_1} \right]^{\frac{-\beta}{b_2}} \right\} \quad (2-14)$$

where

$$vG = \sum_{i=1}^n v_i G_i$$

v_i = Average arrival rate of earthquakes from i th source

n = Total number of sources considered in analysis

G_i = Theoretical geometry factor for i th source that is dependent on whether source is point, line, or area type (see Cornell and Vanmarcke (1969) for a tabulation of G_i for various source types)

There are a number of applications of the variable seismicity approach. Cornell and Vanmarcke (1969) used this technique on a hypothetical site and compared the effects of using a point, line, or area source on the seismic hazard. In this application, the seismic hazard was shown to be relatively insensitive to the upper bound magnitude, M_1 , and most dependent on the more frequent, smaller earthquakes at closer sources. Donovan and Valera (1972) applied the approach to a site located in the San Francisco Bay midway between the Hayward and San Andreas faults and provided results in terms of return periods for different levels of peak acceleration.

2.3.3 DISCUSSION OF APPROACHES

An important advantage of the uniform seismicity approach is that it can be easily applied to an existing site since the theory is relatively simple and the data requirements are typically not difficult to meet. The only data required to implement this method is the recurrence curve for the past earthquake events of the region and the attenuation of intensity with distance for various epicentral intensity levels. This data is usually available from earthquake frequency studies and from isoseismal maps of past earthquake events. Furthermore, the restriction of equal seismicity within the region is not serious for many regions of the United States, which do not exhibit any apparent patterns of markedly higher or lower seismicity.

The variable seismicity approach is more detailed than the uniform seismicity method; its data requirements are more extensive and often must be provided for each potential earthquake source rather than for an entire region. For some regions, this data may be difficult to obtain. For example, the variable seismicity approach requires a description of the geometry, extent, location, and recurrence rates for

each potential earthquake source. The geometry of each source is incorporated in the approach through some theoretically derived parameters for point sources, line sources, or area sources (note the G_1 parameter in Eq. 2-14); the form of these parameters requires verification, possibly using field measurements. In addition, for many regions, such as the eastern and midwestern United States, the extent and location of causative faults (line sources) and the boundaries of tectonic provinces (area sources) cannot be established with certainty. Because of this difficulty, a definitive association of certain earthquake events with one source or another may not always be possible. Finally, the applicability of the variable seismicity method is limited to consideration of those sources for which the available earthquake data are sufficient to define recurrence rates for that source; other potential earthquake sources for which inadequate data are available cannot be incorporated into the variable seismicity approach.

It is noted that both the uniform seismicity method and the variable seismicity approach utilize a deterministic representation of earthquake recurrence rates and intensity-attenuation characteristics. Both approaches should be extended to include probabilistic representations of these parameters.

In summary, the uniform seismicity approach represents a reasonable engineering tool for assessing the seismic hazard at a nuclear plant site, provided the region surrounding the site does not exhibit strong patterns of higher or lower seismicity. The variable seismicity approach can be applied in regions dominated by a number of discrete earthquake sources only if the geometry, extent, and location of all potential earthquake sources can be reasonably well defined, and if sufficient earthquake data can be associated with each source to define meaningful earthquake recurrence rates for that source.

CHAPTER 3

VERTICAL GROUND MOTION CHARACTERISTICS

3.1 INTRODUCTION

Most studies of earthquake-induced ground shaking have dealt with horizontal components of motion, because these components typically represent the primary threat to conventional structures and high-rise buildings. In the past there has been only limited available data from measurements of vertical ground motions, so the basic characteristics of these motions were not well understood. However, numerous studies of nuclear plants have shown that vertical ground motions induced by earthquakes can impart significant dynamic loads to nuclear plant internals, equipment, piping, and support systems; therefore, these motions represent an important design consideration of these facilities. Also, an extensive data-processing program carried out at the California Institute of Technology (CIT) has systematically processed nearly 400 sets of earthquake records (CIT, 1969-1975). The CIT program has provided substantial data from measurements of vertical ground motions, which provides an improved basis for defining vertical input criteria for nuclear plants. This program is described more fully in Chapter 4 below.

An investigation of the characteristics of vertical earthquake ground motions based on the CIT strong motion records is presented here, first assessing the correlation between peak vertical and horizontal ground motions, and second, specifying vertical response spectrum shapes, particularly as they apply to the design of nuclear power plants.

3.2 CORRELATIONS BETWEEN HORIZONTAL AND VERTICAL PEAK GROUND RESPONSES

A series of eight cases was considered for the horizontal-vertical motion correlations, using

four ground response parameters: peak displacement, peak velocity, peak acceleration, and root-mean-square (rms) acceleration. Since each ground motion record contains one vertical and two horizontal components, two ratios (of vertical response to horizontal response) were determined for each parameter. Results were statistically processed, so that the correlations are presented as mean values, standard deviations, and coefficients of variation (the ratio of standard deviation to the mean).

Correlation results are provided in Table 3-1, based on three comparisons (1) all records to those from the 1971 San Fernando earthquake, (2) various strengths of ground shaking, and (3) various local soil conditions. These results are summarized, and current design practice for defining peak vertical acceleration criteria is assessed.

3.2.1 RESULTS OF HORIZONTAL-VERTICAL GROUND RESPONSE CORRELATIONS

The results of the correlations of horizontal and vertical ground responses (see Table 3-1) indicate the following trends:

- a. For a given case, the rms acceleration generally exhibits the smallest overall variability of any of the ground response parameters considered (as indicated by comparisons of the coefficients of variation). The remaining parameters, listed in order of increasing overall data variability, are the peak acceleration, peak velocity, and peak displacement.
- b. The mean value of the rms acceleration is relatively insensitive to the parametric variations considered in each of the eight cases. The peak

TABLE 3-1 SUMMARY OF RESULTS — INVESTIGATION OF CORRELATIONS BETWEEN HORIZONTAL AND VERTICAL GROUND RESPONSES

Category	Case Description	Case Number	Lower Bound for Peak Horizontal Acceleration ¹	Sample Size ²
Comparison of data from all records with data from records measured during 1971 San Fernando earthquake	Data from all records in CIT library	1A 1B	None 0.05g	422 234
	Data from records measured during San Fernando earthquake	2A 2B	None 0.05g	204 146
Assessment of effect of strength of ground shaking (as measured by horizontal peak acceleration, a)	$a \leq 0.10$ g	3A 3B	None 0.05g	268 80
	$0.10 \text{ g} < a \leq 0.20$ g	4	0.10g	118
	$a > 0.20$ g	5	0.20g	36
Assessment of effect of local site conditions at accelerograph station (site classifications defined by Seed, Ugas, and Lysmer, 1974)	Deep soil sites	6A 6B	None 0.05g	102 44
	Stuff soil sites	7A 7B	None 0.05g	32 18
	Rock sites	8A 8B	None 0.05g	50 30

Case Number	Ratio of Vertical Response to Horizontal Response											
	Peak Displacement			Peak Velocity			Peak Acceleration			Root-Mean-Square Acceleration		
	Mean	SD [*]	COV [†]	Mean	SD	COV	Mean	SD	COV	Mean	SD	COV
1A 1B	0.71 0.57	0.52 0.36	0.73 0.63	0.54 0.47	0.27 0.22	0.50 0.47	0.56 0.53	0.28 0.23	0.50 0.43	0.58 0.56	0.19 0.17	0.33 0.30
2A 2B	0.63 0.53	0.46 0.30	0.74 0.57	0.55 0.50	0.26 0.19	0.47 0.38	0.57 0.55	0.22 0.19	0.39 0.35	0.60 0.59	0.17 0.15	0.28 0.25
3A 3B	0.80 0.60	0.58 0.39	0.73 0.65	0.58 0.46	0.28 0.18	0.48 0.39	0.59 0.59	0.29 0.20	0.49 0.34	0.60 0.59	0.20 0.18	0.33 0.31
4	0.55	0.33	0.60	0.48	0.25	0.52	0.53	0.26	0.49	0.56	0.17	0.30
5	0.55	0.38	0.69	0.42	0.22	0.52	0.44	0.16	0.36	0.52	0.12	0.23
6A 6B	0.70 0.55	0.40 0.30	0.57 0.55	0.51 0.43	0.26 0.25	0.51 0.58	0.55 0.50	0.38 0.28	0.69 0.56	0.53 0.52	0.19 0.17	0.36 0.33
7A 7B	0.59 0.44	0.55 0.16	0.93 0.36	0.45 0.38	0.18 0.10	0.40 0.26	0.47 0.45	0.14 0.13	0.30 0.29	0.53 0.51	0.11 0.08	0.21 0.16
8A 8B	0.96 0.88	0.73 0.61	0.76 0.69	0.60 0.52	0.30 0.29	0.50 0.56	0.55 0.52	0.25 0.20	0.45 0.38	0.59 0.58	0.16 0.15	0.27 0.26

*SD = Standard Deviation †COV = Coefficient of Variation (ratio of standard deviation to mean value)

¹For Cases 1 to 3 and 6 to 8, two sets of records were considered. The first considered all of the applicable records in the CIT library, while the second considered only those applicable records whose peak horizontal acceleration exceeded 0.05g.

²Each accelerograph recording consists of two horizontal components of motion and one vertical component. Therefore, two sets of vertical-to-horizontal response ratios can be computed, corresponding to two statistical samples from each accelerograph recording. One or both of these samples may be eliminated for the cases where only records with peak horizontal accelerations greater than 0.05 g are considered.

displacement mean appears most sensitive to these variations.

- c. The data without the lower intensity ground response measurements (i.e., where records having peak accelerations < 0.05 g are deleted) generally exhibit smaller coefficients of variation than the data with these smaller records included. The effects of including data from the lower intensity motions were most significant for peak displacements and least significant for rms accelerations.
- d. Vertical-horizontal response ratios derived from records measured during the San Fernando earthquake (Case 2) are quite similar in magnitude to response ratios derived from the entire set of ground response records in the CIT library (Case 1).
- e. Examination of results from Cases 3 to 5 indicates that, as the peak horizontal acceleration increases, the mean response ratio tends to decrease.
- f. Cases 6 to 8 indicate that the vertical-horizontal ratio of peak displacements is higher for rock sites than for either of the two types of soil sites considered. This trend is also apparent for peak velocities but to a less significant degree than for peak displacements. For mean ratios of peak or rms accelerations, only minor differences exist between rock sites and the two types of soil sites. No consistent trends exist between data from the stiff soil or deep soil sites.

The effects of various parametric variations on the response ratios are indicated in Figures 3-1 through 3-3 for the cases in which the weaker records are deleted. These figures

indicate that:

- a. There is little difference between the cumulative probabilities associated with all of the CIT data (Case 1) and the San Fernando earthquake data (Case 2) (Fig. 3-1).
- b. The cumulative probability of exceedance for a given vertical-horizontal peak acceleration ratio tends to decrease as the intensity of the ground shaking (as represented by the peak horizontal acceleration) increases. This trend is observed to a lesser degree for rms accelerations, and becomes much less clear for peak displacements and peak velocities (Fig. 3-2).
- c. The data from rock sites generally tend to exhibit higher cumulative probabilities of exceedance than do the data from the various soil sites. This trend is strongest for peak displacements and becomes successively weaker for peak velocities, rms accelerations, and peak accelerations. There are no consistent trends regarding the deep soil site data relative to the data from the stiff soil sites. For lower response ratios, the stiff soil site data generally exhibit the higher probabilities whereas at higher response ratios the reverse is true. The response ratio cutoff for this change in trends ranges from about 0.3 for peak displacements to about 0.6 for rms accelerations (Fig. 3-3).

3.2.2 ASSESSMENT OF CURRENT DESIGN PRACTICE FOR DEFINING PEAK VERTICAL ACCELERATION CRITERIA

Current procedures for estimating peak vertical ground acceleration criteria are based

LEGEND	SOURCE OF DATA	NUMBER OF SAMPLES*
————	FROM ALL GROUND MOTION RECORDS PROCESSED AT CIT	234
-----	FROM MOTIONS RECORDED DURING SAN FERNANDO EARTHQUAKE	146

*NOTE: ONLY THOSE RESPONSE RATIOS OBTAINED FROM RECORDS WHOSE PEAK HORIZONTAL ACCELERATION EXCEEDS 0.05 G ARE INCLUDED.

(a) Peak displacements

(b) Peak velocity

(c) Peak accelerations

(d) Root-mean-square accelerations

FIGURE 3-1. CUMULATIVE PROBABILITIES OF EXCEEDANCE FROM ALL GROUND MOTION DATA PROCESSED BY CIT AND FROM RECORDS MEASURED DURING SAN FERNANDO EARTHQUAKE

LEGEND	HORIZONTAL PEAK ACCELERATION, $\ddot{y}_H$	NUMBER OF SAMPLES*
---	$<0.10 \text{ G}$	80
- · - · -	$0.10 \text{ G} < \ddot{y}_H < 0.20 \text{ G}$	118
—	$>0.20 \text{ G}$	36

*NOTE: ONLY THOSE RESPONSE RATIOS OBTAINED FROM RECORDS WHOSE PEAK HORIZONTAL ACCELERATION EXCEEDS 0.05 G ARE INCLUDED.

FIGURE 3-2. EFFECT OF STRENGTH OF GROUND SHAKING ON CUMULATIVE PROBABILITIES OF EXCEEDANCE

LEGEND	SITE CONDITION	NUMBER OF SAMPLES*
————	DEEP SOIL SITE	44
- - - -	STIFF SOIL SITE	18
— · — ·	ROCK SITE	30

*NOTE: ONLY THOSE RESPONSE RATIOS OBTAINED FROM RECORDS WHOSE PEAK HORIZONTAL ACCELERATION EXCEEDS 0.05 G ARE INCLUDED.

(a) Peak displacements

(b) Peak velocities

(c) Peak accelerations

(d) Root-mean-square accelerations

FIGURE 3-3. EFFECT OF SUBSURFACE SITE CONDITIONS ON CUMULATIVE PROBABILITIES OF EXCEEDANCE

on the use of a peak acceleration ratio of 2/3, once the horizontal acceleration criterion is developed. To assess this acceleration ratio, the cumulative probability data in Figures 3-1 to 3-3 can be used to estimate the probability that the 2/3 ratio will be exceeded. These probabilities, from each of the eight cases considered, are summarized in Table 3-2 and indicate that:

- a. When all data in the CIT library are considered, the probability of exceedance of a peak acceleration ratio of 2/3 is 23%. For the remaining cases, the probabilities associated with this ratio range from 8% (for the stiff soil sites in Case 7B) to 34% (for the data with peak horizontal accelerations less than 0.1 g in Case 3B). However, the most typical values for these probabilities are in the range of 20 to 26% (Cases 1, 2, 4, 6, and 8).
- b. For a given case, the probability estimates are typically not strongly affected by whether or not the

weaker records (with peak horizontal accelerations less than 0.05 g) are included in the statistical ensemble. The only exception to this is Case 7, which has the fewest records of any of the various cases considered.

- c. The probabilities of exceedance, as estimated from records measured during the San Fernando earthquakes (Case 2), are only slightly higher than those from all of the data in the CIT library (Case 1).
- d. As the strength of the ground shaking increases (as measured by peak horizontal accelerations) the probability of exceedance tends to decrease. For records whose peak horizontal acceleration exceeds 0.2 g, the probability of exceedance of a vertical-horizontal peak acceleration ratio of 2/3 is only 10%, as compared to 29 to 34% for records whose peak horizontal acceleration is less than 0.1 g (Cases 3, 4, and 5).

TABLE 3-2. ASSESSMENT OF CURRENT DESIGN PRACTICE FOR DEFINING PEAK VERTICAL ACCELERATION CRITERIA

Case Description	Case Number	Probability of Exceedance for Peak Acceleration Ratio of 2/3, %
Data from all records in CIT library	1A	23
	1B	22
Data from records measured during San Fernando earthquake	2A	26
	2B	25
Data with peak horizontal acceleration < 0.10 g	3A	29
	3B	34
Data with peak horizontal acceleration between 0.1 g and 0.2 g	4	20
Data with peak horizontal acceleration > 0.20 g	5	10
Data recorded on deep soil sites	6A	22
	6B	23
Data recorded on stiff soil sites	7A	12
	7B	8
Data recorded on rock sites	8A	25
	8B	22

See Table 3-1 for number of samples and lower bounds for peak horizontal acceleration for each case.

- e. The probabilities associated with the data from deep soil sites and rock sites are nearly equal (22 to 23% vs. 22 to 25% respectively) and are much greater than the probabilities associated with stiff soil sites (8 to 12%) (Cases 6, 7, and 8).

3.3 VERTICAL RESPONSE SPECTRUM SHAPES

Three sets of results are presented for the statistical analyses of vertical response spectrum shapes. The first consists of plots of the various spectra for each case, considering all of the applicable records as well as only those records whose peak vertical accelerations exceed 0.05 g. The second set of results is comprised of overlain plots of the mean spectra and the MSD spectra bounds from the various cases, to determine the effects of each parametric variation on the vertical response spectrum shapes. The plots are based on the data from the records with peak accelerations greater than 0.05 g since (1) the comparative trends regarding the parametric variations do not appear sensitive to the exclusion of the weaker records, and (2) the deletion of the weaker records in these comparisons provides more pertinent results for nuclear plant design. The final set of results consists of comparisons of the composite spectra from each case to the RG 1.60 spectrum shape. Since the RG 1.60 spectra correspond to a mean plus one standard deviation statistical level from the various records considered by Newmark (1973), spectra corresponding to the same statistical level from the present investigation were used in these comparisons.

3.3.1 EFFECTS OF WEAKER RECORDS ON COMPOSITE SPECTRUM SHAPES

The first set of results indicates that the inclusion of the spectra from the weaker records tends to raise the lower frequency end

of the spectrum while not significantly affecting the higher frequency spectral amplitudes (see App. D, AA, 1975c). The highest frequency where the weaker records affect the spectrum shape ranges from about 0.2 Hz (for Cases 6 and 7) to about 4 Hz (for Case 3). Except for Case 3, all of these cutoffs are below 1 Hz, suggesting that the inclusion of the spectra from weaker records does not significantly affect the spectrum shapes in the frequencies of primary interest for nuclear plants (typically, from about 1 Hz to 10 Hz).

Significant scatter in the data is indicated by the differences between the MSD spectra bounds (see App. D, AA, 1975c). Furthermore, the MSD spectra bounds that correspond to inclusion or deletion of the spectra from the weaker records for each case, tend to overlap significantly; this suggests that the trends indicated above may often be obscured by data scatter. Such scatter is seen to be reduced substantially by deleting the spectra from the weaker records; however, in all cases, it is sufficiently significant to warrant careful consideration when interpreting trends from the statistical analyses.

3.3.2 EFFECTS OF PARAMETRIC VARIATIONS ON COMPOSITE SPECTRUM SHAPES

To evaluate the effects of the parametric variations associated with the individual cases, three sets of comparisons of mean and MSD spectra bounds were carried out. The first (see Fig. 3-4) considers a comparison of the spectra from all records in the CIT library (Case 1) to spectra from records obtained during the San Fernando earthquake (Case 2). The second comparison (see Fig. 3-5) shows the effect of the peak vertical acceleration level on the vertical response spectrum shape; this figure consists of overlain composite spectra from the various peak acceleration ranges considered in Case 3 (0.05 to 0.10 g), Case 4 (0.1

to 0.2 g), and Case 5 (>0.2 g). The final comparison (see Fig. 3-6) indicates the effect of subsurface site conditions on the composite spectrum shape, and provides overlays from the spectra developed from Case 6 (deep soil), Case 7 (stiff soil) and Case 8 (rock). These comparisons indicate the following trends:

- a. In Figure 3-4, the spectra are nearly identical at frequencies above 1 Hz and there is only slight variation for frequencies below 1 Hz. The CIT library of records, therefore, do not appear to be biased by the great number of San Fernando earthquake records.
- b. In Figure 3-5, nearly identical spectra were obtained for frequencies above 4 Hz. Reasonable agreement was obtained below 4 Hz for the two lowest acceleration levels (Cases 3 and 4), but the spectrum for the highest acceleration level departs significantly at the lower frequencies. This may be partly due to the bias of the relatively small sample (12 records) used for Case 5, but most of the low frequency shift is probably due to the fact that peak acceleration normalization procedures were used prior to statistical processing. It is recommended that normalization procedures be studied that use different factors (i.e., peak acceleration, peak velocity, and peak displacement) for different segments of the spectral curves.
- c. In Figure 3-6, no significant difference in spectra due to subsurface conditions could be detected for frequencies above 3 Hz. Below 3 Hz, no significant trend could be detected

between deep soil and stiff soil conditions, except greater scatter in data for the former. The response of rock sites below a frequency of 3 Hz, however, was significantly below that for soil sites.

3.3.3 COMPARISONS OF COMPOSITE SPECTRUM SHAPES WITH RG 1.60

Normalized spectra from the eight cases considered here are compared with RG 1.60 in Figure 3-7. These comparisons are based on spectra corresponding to a mean plus one standard deviation statistical level for each case. The following trends are indicated:

- a. The spectra developed for each of the eight cases fall well below the RG 1.60 spectrum in the frequencies of interest for nuclear plant design - 1 to 10 Hz. These differences, as much as 50 to 100% in many instances, indicate that the RG 1.60 provisions for vertical response criteria are overly conservative for design purposes.
- b. The comparisons of the statistically derived spectra with RG 1.60 show somewhat different comparisons for frequencies below 1 Hz. At these lower frequencies, and when spectra from the weaker records are excluded, the RG 1.60 spectrum compares closely with the spectra from Cases 1, 2, 3, 4, and 7. The lower frequency spectral amplitudes from Case 5 (peak accelerations > 0.2 g) and from Case 8 (rock sites) fall below those of RG 1.60. Only for Case 6 (deep soil sites) do the low frequency spectral amplitudes exceed RG 1.60.

c When the spectra from the weak records are included, the lower frequency amplitudes of the statistically derived spectra are increased, but the higher frequency (above about 3 Hz) amplitudes are relatively

unaffected Part of the shift is probably again due to the normalization procedures used The amplitudes at the low frequency end of all spectra are therefore probably high

(a) Mean spectra

(b) MSD bounds

FIGURE 3-4 COMPARISON OF FIVE-PERCENT DAMPED SPECTRUM SHAPES FROM ALL CIT RECORDS TO THOSE FROM SAN FERNANDO EARTHQUAKE RECORDS

(a) Mean spectra

(b) MSD bounds

FIGURE 3-5. COMPARISONS OF FIVE-PERCENT DAMPED SPECTRUM SHAPES CORRESPONDING TO DIFFERENT PEAK ACCELERATION LEVELS

(a) Mean spectra

(b) MSD bounds

FIGURE 3-6. COMPARISONS OF FIVE-PERCENT DAMPED SPECTRUM SHAPES CORRESPONDING TO DIFFERENT SITE CONDITIONS

(a) All records in CIT library (Case 1)

(b) Records measured during San Fernando earthquake (Case 2)

FIGURE 3-7. COMPARISONS OF VERTICAL RESPONSE SPECTRUM SHAPES FROM RG 1.60 AND FROM PRESENT INVESTIGATION

(c) Records whose peak acceleration is less than 0.1 g (Case 3)

(d) Records whose peak acceleration is between 0.1 g and 0.2 g (Case 4)

FIGURE 3-7. COMPARISONS OF VERTICAL RESPONSE SPECTRUM SHAPES FROM RG 1.60 AND FROM PRESENT INVESTIGATION (Continued)

(e) Records whose peak acceleration exceeds 0.2 g (Case 5)

(f) Records measured on deep soil sites (Case 6)

FIGURE 3-7. COMPARISONS OF VERTICAL RESPONSE SPECTRUM SHAPES FROM RG 1.60 AND FROM PRESENT INVESTIGATION (Continued)

(g) Records measured on stiff soil sites (Case 7)

(h) Records measured on rock sites (Case 8)

FIGURE 3-7. COMPARISONS OF VERTICAL RESPONSE SPECTRUM SHAPES FROM RG 1.60 AND FROM PRESENT INVESTIGATION (Concluded)

CHAPTER 4

VERTICAL GROUND RESPONSE SPECTRUM SHAPES

The investigation described here considers three groups of records contained in an overall ensemble of 186 vertical free-field records, in order to:

1. Compare various procedures for normalizing vertical motion data when developing composite spectrum shapes for design purposes, and select a preferred procedure for developing such spectrum shapes
2. Compare these spectrum shapes to those based on current design practice from RG 1.60
3. Determine effects of local soil conditions and geology on vertical motion spectrum shapes

4.1 BACKGROUND

Most studies of earthquake-induced ground shaking have dealt with only the horizontal components of motion, because these motions represent the primary threat to the integrity of conventional structures. Also, until recently, there has been only limited available data from measurements of vertical ground motions; as a result, the basic characteristics of these motions were not well understood.

In recent years, two factors have changed this picture. First, studies of nuclear power plants have shown that vertical motions can impart significant dynamic loads to nuclear plant internals, equipment, piping, and support systems; therefore such motions require careful consideration in the design. Second, an extensive data processing program carried out at the California Institute of Technology (CIT) over several years has systematically processed nearly 400 sets of earthquake records that have

been measured in buildings and in the free field since 1933; this program has included the processing of 186 vertical free-field records, which represents a much improved data base for studying the characteristics of vertical ground motions.

4.2 DATA DESCRIPTION AND ANALYSIS PROCEDURE

4.2.1 DATA BASE

The CIT data processing program was initiated in 1969 to provide strong motion data that is processed in a uniform manner and as accurately as possible. The program is described in the various CIT reports containing the data (CIT, 1969-1975) and in various papers (e.g., Trifunac, Udawadia, and Brady, 1973).

The procedure followed by CIT first involves a digitization of the field accelerograph records using semiautomatic or completely automatic image scanning processes (Hudson, 1974; 1979). Two basic sets of numerical corrections are then applied to the digitized data: an instrument correction and a baseline correction. The instrument correction consists of a low-pass filtering of the original data to eliminate high-frequency digitization errors and aliasing; then, a differential equation of motion for a damped single-degree-of-freedom system with a high natural frequency is applied to the data to recover higher frequency components of motion that have been filtered by the transducer (Trifunac, 1972c). The baseline correction consists of the use of a high-pass filter that filters out periods longer than 16 sec and eliminates the long period errors introduced by recording, digitization, and record processing

(Trifunac, 1971). The final result is a corrected accelerogram that represents the acceleration of the instrument base within a frequency range of 0.06 Hz to 25 Hz. These accelerograms are integrated to produce ground velocity and displacement records, and response spectra and Fourier spectra in various forms are also calculated.

This procedure has been applied to numerous records from 57 earthquakes in the western United States and predominantly in California, with nearly half of these records originating from a single event – the 1971 San Fernando earthquake. The resulting data are available in the form of digitized printouts and plots in the CIT reports, and are also provided on magnetic tapes or punched cards that can be purchased from CIT or various government agencies. To date, such data have been developed for nearly 400 sets of strong motion records, each consisting of three accelerogram components. Of these, 186 sets of records correspond to measurements on the ground surface or in lower floors and basements of buildings, and are generally considered representative of free-field ground shaking. The remaining records represent measurements in the mid-to-upper floors of buildings; these records are influenced by the dynamic response characteristics of the structures, as well as by the ground motions themselves.

The data used in this investigation consist of response spectra developed from various groups of the 186 vertical ground motion records that were processed by CIT. Each of these spectra has been defined on magnetic tapes obtained from CIT in terms of pseudo-velocity amplitudes at 91 discrete frequencies between 0.06 Hz and 25 Hz. Five spectra have been developed for each record, corresponding to damping ratios of 0.0, 0.02, 0.05, 0.10, and 0.20. The entire set of records has been tabulated (see App. A, AA, 1977).

4.2.2 DATA CLASSIFICATION

Statistical analyses for this study have been based on various vertical motion record ensembles that are defined from three main classifications. The first consists of all records whose peak accelerations exceed 0.03 g, while the second and third classifications are based on two different approaches for representing local site conditions. These classifications are enumerated in Table 4-1.

The first ensemble (Category 1 in Table 4-1) is used for (1) the assessment of various methods for normalizing vertical motion data when developing spectrum shapes; (2) the development of vertical response spectrum shapes for various damping ratios; and (3) the comparison of these spectrum shapes with those from RG 1.60. It consists of all vertical records processed by CIT, regardless of site condition, that have a peak acceleration that exceeds 0.03 g. This acceleration cutoff was selected because it corresponds to the lowest peak vertical acceleration that has been considered acceptable in the development of seismic input criteria for nuclear plants (USAEC, 1973a). Also, the inclusion of weaker records influences the resulting spectrum shape (see Chapter 3 above). Therefore, since such records are not applicable in nuclear plant design, they have been eliminated from inclusion in this ensemble. On this basis, the ensemble includes 112 of the 186 vertical motion records in the CIT library.

Category 2 assesses the sensitivity of vertical response spectrum shapes to subsurface soil conditions, based on the premise that the subsurface soil and rock materials within a few hundred feet of the ground surface have a significant influence on the earthquake motion measurements (Seed, 1969; Schnabel et al., 1972). The data are classed according to three

TABLE 4-1. ENSEMBLES OF VERTICAL GROUND RESPONSE DATA

- Category 1 All data, regardless of site condition, with peak vertical acceleration > 0.03 g
- Category 2 Data classed according to subsurface soil conditions ¹
- Category 3 Data classed according to local geologic conditions ²

Category	Damping Ratios	Site Condition	Sample Size	Composite Peak Vertical Motions for Ensemble			
				Probability Level ³	Acceleration, g's	Velocity, ips	Displacement, in
1	0.0 0.02 0.05 0.10 0.20	--	112	Mean	0.074	2.81	1.89
				84th Percentile	0.108	4.14	2.70
2	0.05	Rock	15	Mean	0.112	3.18	2.19
				84th Percentile	0.184	5.09	3.33
		Stiff Soil	49	Mean	0.057	2.48	2.17
				84th Percentile	0.092	3.41	3.21
		Deep Soil	66	Mean	0.055	2.67	2.10
				84th Percentile	0.091	3.94	2.94
3	0.05	Basement Rock	15	Mean	0.113	3.41	2.29
				84th Percentile	0.186	5.49	3.52
		Intermediate Rock	54	Mean	0.046	2.20	2.26
				84th Percentile	0.076	3.11	3.30
		Alluvium	117	Mean	0.053	2.52	2.04
				84th Percentile	0.087	3.65	2.94

¹Subsurface soil conditions categorized according to Seed-Ugas-Lysmer (1976) classification

²Local geologic conditions categorized according to Trifunac-Brady (1975) classification

³Probability levels based on assumed lognormal distribution

general types* of site soil conditions (Seed, Ugas, and Lysmer, 1976)

Deep Soil Sites – Soil deposits whose depth to rock exceeds 250 ft

Stiff Soil Sites – Firm soil deposits whose depth to rock is less than 150 ft

Rock Sites – Located on rock outcropping

Accelerograph site soil classifications were based on past studies (SW/AA, 1976) or the best available information. Those sites for

which no reliable geotechnical information could be obtained were not assigned a site soil classification. On this basis, 130 of the 186 vertical motion records in the CIT library were classified into one of the above three site categories (see Table 4-1).

*A fourth classification is soft-to-medium stiff clays and sands, but none of the sites considered here are in this category.

Category 3 considers that the recorded ground motions have been influenced by subsurface materials that extend to depths comparable to the wavelengths of the significant frequencies of the motions, which means characteristics at depths far greater than those considered in Category 2.* This is a local geologic classification, based on geologic maps and other site geology data (Trifunac and Brady, 1975; Wong and Trifunac, 1977), rather than typical site soils investigations (SW/AA, 1972). The general site classifications are (Trifunac and Brady, 1975):

Soft alluvium deposits

Intermediate (or hard sedimentary) rock

Basement (or crystalline) rock

Each of the 186 vertical motion records has been assigned one of these three classifications, based on an assessment at CIT (see Table 4-1).

4.2.3 STATISTICAL ANALYSIS PROCEDURE

Standard statistical analysis procedures are used on the vertical ground response spectra. For each ensemble of data, the pseudovelocities at a specified frequency are first normalized with respect to the appropriate peak motion parameter. These normalized pseudovelocities are treated as lognormally distributed random variables (see Chapters 3 and 6; also Hall, Mohraz, and Newmark, 1976), and the ensemble mean, standard deviation, and other pertinent information are derived at each of the 91 frequency points at which spectral amplitudes are defined on the CIT data tapes. This information can then be used to construct vertical

ground response spectrum shapes that correspond to any desired probability level.

The normalized spectrum shapes are then scaled using an appropriate peak motion parameter. To obtain parameter values for this purpose, the peak vertical acceleration, velocity, and displacement for the records in each data grouping are treated as lognormally distributed random variables. Ensemble means and standard deviations are obtained for each parameter so that values at any probability level can be defined. Values of the peak motions that correspond to two particular probability levels – the ensemble mean and 84th percentile – are used herein to scale the spectrum shapes to obtain vertical ground response spectra.

To evaluate the results using normalized spectrum shapes, spectra developed from natural data, or data that has not been first normalized to some peak motion parameter, are used. The procedure for operating on such data to obtain composite spectra that represent a particular probability level is identical to that described above for the normalized spectrum shapes, except that the normalizing operation is deleted.

4.3 ANALYSIS RESULTS

4.3.1 DESCRIPTION OF NORMALIZING METHODS

There are two basic procedures for developing composite ground response spectra from existing strong motion records. The most widely used approach is based on the use of spectra that are first normalized according to some peak motion parameter – peak acceleration, peak velocity, and peak displacement. Such normalized spectra, developed for each record in the ensemble, are analyzed statistically to obtain composite spectrum shapes that represent

*Wong and Trifunac (1977) indicate that for frequencies ranging from 0.1 Hz to 20 Hz and for wave speeds ranging from 0.1 km/sec to 3 km/sec, the range of wavelengths (and corresponding depths of subsurface materials) to be considered is from 5 m to 30 km.

some prescribed probability of exceedance. These spectrum shapes are then scaled according to a given peak motion level to obtain the design spectrum for the site.

The main advantage of this approach is its simplicity, particularly when standardized spectrum shapes advocated by regulatory agencies are available and appropriate for use (e.g., RG 1.60). For this case, the engineer is required only to determine the peak motions required to scale these shapes when defining criteria spectra. The disadvantage of this use of normalized spectra is that the normalization process distorts the spectrum shape and leads to data scatter that varies significantly with frequency. Also, composite spectra developed using this approach cannot exactly represent a given probability level without significant difficulty; instead, individual probability levels associated with both the spectrum shape and with the peak motions must be combined in an approximate manner to estimate a composite overall probability.

The second basic procedure for developing ground response spectra uses data from measured records that have not first been normalized, herein termed "natural data." This procedure corresponds to statistical analyses of spectra determined directly from those vertical ground motion records that have a peak acceleration greater than 0.03 g. In this investigation, no attempt is made to group the natural data according to different ranges of strength levels; this is consistent with most past practice regarding the development of composite spectra from normalized data (e.g., RG 1.60) and with the way the normalized data are used to develop composite spectra in the first approach described above. However, it should be noted that a number of investigations have distinguished between various strength levels when developing spectra from natural data (Trifunac, 1977; Chapter 6 below).

The natural data results are used herein as a basis for comparison with the various normalized spectra, which are the main focus of this investigation. However, natural data have several advantages of their own when applied to develop vibratory motion criteria spectra. First, they provide spectra that are not distorted by the normalization process. Also, spectra can be readily defined that correspond to a given probability of exceedance, without the approximations required when normalized spectra are used.

Four different methods for normalizing data when developing vertical response spectrum shapes are considered here. The first three methods are based on data normalized according to a single peak motion parameter over the entire frequency range. The fourth method – termed the three-segment method – uses all three peak motion parameters to normalize the data over an appropriate segment of the overall frequency range. Results from these four methods are compared to each other and to spectra derived from natural data in the following section.

Method 1 – Data Normalized to Peak Acceleration

For nuclear plant design, seismic input criteria are most typically based on statistical analyses of records normalized to a unit peak acceleration (e.g., RG 1.60). This approach for developing spectrum shapes has been used to provide a consistent basis for comparison with the RG 1.60 spectrum shapes (Chapter 3 above). However, the normalization of records in this manner resulted in composite spectrum shapes with very small standard deviations at high frequencies and very large standard deviations at low frequencies (see also, Hall, Mohraz, and Newmark, 1976).

Method 2 – Data Normalized to Peak Velocity

A second approach for developing vertical

response spectrum shapes is based on statistical analyses of records that have first been normalized to a unit peak velocity. Various investigators have advocated the use of the peak velocity to represent the strength of the ground shaking in this manner, on the basis that it is a more stable parameter with a more determinant upper bound than the peak acceleration (e.g., Newmark and Hall, 1969). Also, it has been shown that spectrum shapes from records normalized according to peak velocity have a standard deviation that is much more uniform over the entire frequency range of interest than that from data normalized according to peak acceleration or displacements (Hall, Mohraz, and Newmark, 1976).

Method 3 – Data Normalized to Peak Displacement

A third method is based on the statistical analyses of spectra from vertical motion records that have first been normalized according to a unit peak displacement. This approach, along with peak velocity normalizing, has been advocated by some investigators as an improvement over procedures involving normalization according to peak acceleration (Hudson, 1972). However, the approach has the disadvantage that peak displacements, determined by doubly integrating the acceleration records, are prone to uncertainties in long-period response introduced by the recording and processing of the accelerograms (Trifunac, 1971). Also, Hall, Mohraz, and Newmark (1976) have shown that spectra based on records normalized according to peak displacement have a standard deviation that increases from a very small value at low frequencies to a relatively large value at high frequencies.

Method 4 – Three-Segment Method

A fourth method, outlined in Figure 4-1, involves the following steps:

- a. Normalize the response spectrum from each record in the ensemble in three different ways – according to peak acceleration in the high-frequency range (1 Hz to 25 Hz), to peak velocity at intermediate frequencies (from 0.25 Hz to 5 Hz), and to peak displacements in the low frequency range (from 0.125 Hz to 1 Hz).
- b. Carry out statistical analyses of these three sets of data to obtain composite normalized spectrum shapes for each frequency range. These spectrum shapes represent a given prescribed probability of exceedance, p_1 .
- c. Perform statistical analyses of the ensemble of ground motion records to obtain a peak acceleration, velocity, and displacement that correspond to a desired probability of exceedance, p_2 .
- d. Use these peak motions to scale the normalized spectrum shapes in the high, intermediate, and low frequency range respectively. This results in three separate spectra defined for the three frequency ranges.
- e. Since the three frequency ranges overlap one another (see Step a), interconnect the three separate spectra by smooth transition curves (Brady, 1975). This forms a single composite spectrum that represents two different probability levels – one from the normalized spectrum shapes (denoted in Step b as p_1), and the other from the peak motions (indicated as p_2 in Step c).

The advantage of this method is that it is based on spectrum shapes that are normalized according to the peak motion parameter that has the greatest influence on the response spectrum in

FIGURE 4-1. THREE-SEGMENT METHOD FOR OBTAINING RESPONSE SPECTRA (Brady, 1975)

each frequency range. Furthermore, the steps are straightforward and no more difficult than those of the other methods.

4.3.2 COMPARISON OF RESULTS

Mean and 84th percentile spectra developed using the four normalization methods are compared to each other and to results from the natural data in Figure 4-2, with the following results:

- The mean and 84th percentile spectra developed using the three-segment approach (Method 4) fall quite close to those from the natural data, except at very low frequencies below about 0.1 Hz.
- The mean and 84th percentile spectra from the data normalized according to peak acceleration (Method 1) also fall

quite close to those from Method 4 throughout the entire frequency range shown

- The mean and 84th percentile spectra from the data normalized according to peak velocity (Method 2) fall slightly above the spectra from the natural data and from Methods 1 and 4 at frequencies greater than about 1 Hz and fall slightly below the spectra from those methods at frequencies below 1 Hz. The differences between the Method 2 spectra and those from the natural data and from Methods 1 and 4 are less than about 25% at all frequencies.

- The spectra from the data normalized according to peak displacement (Method 3) fall well above the spectra from the other methods at frequencies above about 0.2 Hz.

In summary, the mean and 84th percentile vertical ground response spectra developed from the natural data, from the three-segment method (Method 4), and from the data normalized according to peak velocity and acceleration (Methods 1 and 2) are generally quite similar to one another. The spectra from the data normalized according to peak displacement (Method 3) exceed those developed from the other methods over nearly the entire frequency range of interest.

LEGEND	DATA PROCESSING METHOD		BASIS FOR SCALING COMPOSITE SPECTRUM SHAPE
	METHOD NO	DESCRIPTION	
—	--	NATURAL DATA	NO SCALING
---	1	NORMALIZED TO PEAK ACCELERATION, THEN ANALYZED STATISTICALLY	ENSEMBLE MEAN VALUE OF PEAK ACCELERATION
- - -	2	NORMALIZED TO PEAK VELOCITY, THEN ANALYZED STATISTICALLY	ENSEMBLE MEAN VALUE OF PEAK VELOCITY
- - -	3	NORMALIZED TO PEAK DISPLACEMENT, THEN ANALYZED STATISTICALLY	ENSEMBLE MEAN VALUE OF PEAK DISPLACEMENT
.....	4	THREE-SEGMENT METHOD	ENSEMBLE MEAN VALUE OF PEAK DISPLACEMENT, VELOCITY, AND ACCELERATION

(a) Mean spectral amplification curves

(b) 84th percentile spectral amplification

FIGURE 4-2. EFFECT OF DATA-PROCESSING METHOD ON SPECTRAL AMPLIFICATION CURVES FOR VERTICAL GROUND RESPONSE

These results can be used to select a preferred method for developing damped vertical response spectra. The approach involving the normalization of the data according to peak acceleration (Method 1) provided mean and 84th percentile composite spectra that compared surprisingly well with those from the natural data; however, Method 1 has the disadvantage of resulting in scatter bands that vary significantly with frequency, with the greatest scatter observed for the low-frequency spectral amplitudes. Method 3, which involves normalizing of the data according to peak displacement, appears to distort the higher-frequency spectral amplitudes, as noted from the comparisons of the mean and 84th percentile spectra in Figure 4-2.

The method involving normalization of the data according to peak velocity (Method 2) and the three-segment method (Method 4) are judged to be most favorable, based on the comparisons with the spectrum shapes and scatter bands exhibited by the natural data. However, Method 2 exhibits somewhat wider overall scatter bands and provides spectrum shapes that differ from those of the natural data to a greater extent than do the spectra from the three-segment method. Furthermore, the three-segment method has the advantage of normalizing the data in each frequency range according to the peak motion parameter that has the greatest influence on the spectral amplitudes in that range. For these reasons, the three-segment method was selected for use in the subsequent development of vertical ground response spectra.

4.3.3 DEVELOPMENT OF VERTICAL GROUND RESPONSE SPECTRA

Vertical ground response spectra for five different damping ratios – corresponding to 0, 2, 5, 10, and 20% of critical – are shown in Figure 4-3. These damping ratios correspond

to those for which spectra have been developed for the individual records by CIT (1969-1975) and are provided on the CIT data tapes. The four parts of Figure 4-3 present four sets of damped spectra. The first corresponds to mean spectra, approximated by using the mean spectrum shape in each frequency range, scaled according to the ensemble mean value of the peak displacement, velocity, and acceleration. The remaining sets of spectra correspond to various approximations of the 84th percentile probability level. Part b contains spectra obtained by using the 84th percentile peak motions to scale the mean spectrum shapes, and Part c provides spectra developed by using 84th percentile spectrum shapes scaled using the mean values of the peak motions. These two sets of spectra are very similar to one another. The 84th percentile spectrum is provided in Part d, and the resulting spectra are obviously too high.

Comparisons of vertical ground response spectra developed using the three-segment method and RG 1.60 procedures are shown in Figure 4-4. Two sets of comparisons are presented, one in which the spectra are scaled to mean values of peak motions and the other in which 84th percentile peak motion values are used to scale the spectra. For each peak motion case, spectra are developed using three-segment-method spectral amplification factors that represent mean and 84th percentile probability levels. All probability levels are based on the 112 records contained in Category 1 of Table 4-1.

The RG 1.60 vertical ground response spectra are based on spectral amplification factors that are multiples of the peak horizontal acceleration. Therefore, to develop RG 1.60 spectra that are consistent with the above peak motion cases, statistical analyses of the horizontal components of motion that correspond

LEGEND	SPECTRAL DAMPING RATIO
————	0.00
-----	0.02
- - - -	0.05
_____	0.10
- . - . -	0.20

(a) Mean spectral amplification curves scaled to mean values of peak motions

(b) Mean spectral amplification curves scaled to 84th percentile values of peak motions

(c) 84th percentile spectral amplification curves scaled to mean values of peak motions

(d) 84th percentile spectral amplification curves scaled to 84th percentile values of peak motions

FIGURE 4-3. DAMPED VERTICAL GROUND-RESPONSE SPECTRA DEVELOPED USING THREE-SEGMENT METHOD

(a) Spectra scaled to mean values of peak motions

(b) Spectra scaled to 84th percentile values of peak motions

FIGURE 4-4. COMPARISONS OF VERTICAL GROUND RESPONSE SPECTRA FROM REGULATORY GUIDE 1.60 AND THREE-SEGMENT METHOD

to the 112 vertical records included in Category 1 of Table 4-1 were carried out. Peak horizontal accelerations corresponding to the mean and 84th percentile probability levels from these horizontal records were thereby obtained and were used to scale the RG 1.60 spectrum shape (see Fig. 4-4). Therefore the comparisons based on mean values of peak motions involved the RG 1.60 spectrum shape scaled to the mean value of the peak horizontal acceleration, and the three-segment method spectrum shapes scaled to mean values of the peak vertical acceleration, velocity, and displacement. Corresponding comparisons were made for the case involving the 84th percentile peak motions.

Results of these comparisons (Fig. 4-4) indicate that the RG 1.60 spectra exceed the spectra from the three-segment method, often by a significant amount. This trend holds regardless of which probability level is used for the peak motions or for the spectral amplification factors in the three-segment method. This has been observed in other studies of vertical motion records involving other approaches for processing the data (see Chapters 3 and 6), which suggests that the RG 1.60 vertical response spectra are overly conservative for use in design applications.

4.3.4 LOCAL SITE EFFECTS ON VERTICAL GROUND RESPONSE SPECTRUM SHAPES

4.3.4.1 Background

Several studies of the effects of local site conditions on response spectra and spectrum shapes are based on the subsurface soil data classification and have developed spectrum shapes derived from records normalized according to peak ground accelerations (Seed, Ugas, and Lysmer, 1976; also Chapter 3 above). Other studies have departed from this approach. For example, Mohraz (1976) computed the statistics of the ratio of the velocity

to the acceleration, v/a , and the ratio of the acceleration-displacement product to the square of the velocity, ad/v^2 . These ratios were used to define the peak displacement and velocity from a given peak acceleration and these three parameters were, in turn, used to scale spectral amplification factors in the low, intermediate, and high frequency ranges from data classed according to subsurface soil conditions.* Other departures from the practice of considering spectra normalized according to peak accelerations only include Trifunac (1977) and Chapter 6 below. Trifunac used regression analysis procedures to develop a statistical model that defined horizontal and vertical ground response spectral amplitudes for given probability levels, local geologic conditions, and site intensities. The approach given in Chapter 6 carried out statistical analyses of horizontal and vertical ground response spectra from measured records classed according to site intensity and, for each intensity, according to subsurface soil and local geologic conditions.

Results from these studies are somewhat varied, probably reflecting differences in the data base, in the way that individual records were classed according to local site conditions, and in data-analysis methods. The Seed-Ugas-Lysmer study showed significant differences between the normalized horizontal response spectra developed for each subsurface soil condition. Significant differences between the rock site vertical spectra relative to those for the stiff soil and deep soil site conditions for frequencies below about 4 Hz are given in Chapter 3 above; however, the composite spectra for the stiff soil and deep soil sites were

*Mohraz assumes the peak displacement and velocity to be dependent on peak acceleration, in his use of the v/a and ad/v^2 ratios. In contrast, the three-segment method considers each of these peak motion parameters to be independent.

quite similar to one another over the entire frequency range. Trifunac's statistical model showed local geologic effects that were not always significant, particularly when the intermediate rock condition was included. The study in Chapter 6 indicated no statistically significant and consistent differences in spectra due to the different site conditions considered.

4.3.4.2 Present Investigation

This investigation of local site effects on response spectrum shapes used data processed by the three-segment method. The appropriate ensemble of vertical motion records was first subdivided into one of the prescribed site condition classifications (see Table 4-1). The spectra from the records grouped into each classification were normalized with respect to peak displacement, velocity, and acceleration and then analyzed statistically. Results from this analysis were provided in the form of normalized composite spectrum shapes that represent specified probability levels. The spectrum shapes normalized to peak displacement were provided for the low-frequency range (0.125 Hz to 1 Hz), the spectrum shapes normalized to peak velocity in the intermediate-frequency range (0.25 Hz to 5 Hz), and those normalized to peak acceleration in the high-frequency range (1 Hz to 25 Hz). Within each frequency range, the normalized spectra for each site classification were compared, to indicate the influence of local site conditions in that range. The comparisons were carried out for a spectral damping ratio of 5% of critical and for mean and 84th percentile probability levels.

As previously described, the three-segment method includes a scaling of the normalized spectral amplitudes according to the appropriate peak motion parameter for each frequency range; in this way, the total composite spectrum is obtained. However, this scaling

procedure has been deleted from the following comparisons of local site effects, since the peak motions are strongly affected by other factors besides local site conditions, such as the size of the earthquake, the distance from the site to the center of energy release, etc. Although these other factors undoubtedly affect the normalized spectral amplitudes as well, it is judged that their effects on these normalized amplitudes will be much less than their influence on the peak motions. Therefore, the effects of local site conditions are isolated here and evaluated by comparing only the normalized spectral amplitudes within each frequency range, rather than the total scaled spectrum.

How should the site-dependent normalized spectral amplitudes compare, if local site effects are significant? Prior studies indicate that the deepest and softest site conditions should have the largest amplifications at low frequencies and the smallest amplifications at high frequencies. Conversely, the hardest site conditions should exhibit the greatest high-frequency spectral amplification factors and the smallest low-frequency factors. The intermediate site conditions should provide amplification factors that fall within these two extremes.

Results from the present investigation are shown in Figures 4-5 and 4-6 for the subsurface soil and local geologic classifications. These are described separately for the data normalized according to peak acceleration, peak velocity, and peak displacement.

- Peak Acceleration. Spectral amplitudes that represent frequencies ranging from 1 Hz to 25 Hz were normalized according to peak acceleration. For this case, the results from data classed according to subsurface soil conditions show comparisons that are generally consistent with those anticipated above for conditions where

(a) Ensemble mean normalized spectral amplitudes

(b) Ensemble 84th percentile normalized spectral amplitudes

FIGURE 4-5. EFFECTS OF SUBSURFACE SOIL CONDITIONS ON NORMALIZED SPECTRAL AMPLITUDES

(a) Ensemble mean normalized spectral amplitudes

(b) Ensemble 84th percentile normalized spectral amplitudes

FIGURE 4-6. EFFECTS OF LOCAL GEOLOGIC CONDITIONS ON NORMALIZED SPECTRAL AMPLITUDES

local site effects are significant (Fig. 4-5). Below about 4 Hz, the rock site conditions exhibit the lowest spectral amplitudes and the deep soil sites exhibit the greatest amplitudes. Above about 8 to 10 Hz, the opposite trend holds, i.e., the rock sites exhibit slightly greater amplitudes than the stiff soil sites, and the deep soil sites show the lowest amplitudes. The stiff soil sites generally represent an intermediate condition between the rock and deep soil sites, except between about 4 to 8 Hz which appears to be a transition region for these normalized spectrum shapes.

The results from data classed according to local geology and normalized according to peak acceleration (Fig. 4-6) exhibit trends slightly different from those noted above. The primary difference is that there is no clear distinction between the spectral amplitudes for the alluvium and intermediate rock sites. However, the differences in amplitudes for these two local geologic conditions relative to the amplitudes for the basement rock sites are consistent with those noted for the data classed according to subsurface soil conditions.

- Peak Velocity. The normalization of spectral amplitudes according to peak velocity was carried out within the frequency range from 0.25 Hz to 5 Hz. For this case, results for the data classed according to subsurface soil conditions (Fig. 4-5) show virtually no consistent trends regarding local site effects. At frequencies below about 1 Hz, the stiff soil sites show the greatest spectral amplitudes followed, in order, by the deep soil

sites and the rock sites. Above 1 Hz, there is virtually no distinction between the three sets of normalized spectral amplitudes.

When the data are classed according to local geology, the trends are similar to those noted above, i.e., essentially no distinct local site effects on the spectral amplitudes can be observed. Below about 0.6 Hz, the alluvium sites exhibit the largest amplitudes, followed by the intermediate rock and basement rock sites. Above about 2 Hz, the alluvium sites show the smallest amplitudes and the spectral amplitudes for the intermediate rock sites and basement rock sites are nearly identical. Between 0.6 Hz and 2 Hz, which constitutes the largest part of the frequency range over which the data are normalized according to peak velocity, there is virtually no distinguishable effect of local site conditions on the normalized spectral amplitudes.

- Peak Displacement. At frequencies ranging from 0.125 Hz to 1 Hz, the spectral data were normalized according to peak displacement. Throughout this frequency range, and regardless of whether the data are classed according to subsurface soil conditions or local geology, there are no clear effects of local site conditions on the spectral amplitudes. In the subsurface soil classification, for example, the spectral amplitudes for rock and deep soil sites are nearly identical to one another, except at very low frequencies; furthermore, the stiff soil sites do not represent an intermediate condition between these two extremes. For data classed

according to local geology, there are isolated instances where faint local site effects can be observed; however, the overall trend for these results is that local site effects are indistinguishable.

4.3.4.3 Conclusions

Of particular importance in the results presented here is how local site effects are dependent on how the data are normalized. In the investigation reported in Chapter 3 of this volume, vertical ground response spectrum shapes were based on spectra normalized according to peak acceleration over the entire frequency range of interest and showed important differences between lower frequency spectral amplitudes for the rock sites relative to those for stiff soil and deep soil sites. Similar trends are indicated by the results of the present investigation for the data normalized according to peak acceleration. In fact, other studies showing important site-dependent effects have also been based on data normalized to a

given peak acceleration level (e.g., Seed, Ugas, and Lysmer, 1976).

In contrast, spectra based on data normalized according to peak velocity or peak displacement show no consistent site-dependent results. The comparison of results from data normalized according to peak velocity and to peak acceleration, where the frequency ranges for the two sets of spectra overlap (1 to 5 Hz), indicates the importance of the data normalizing method; i.e., for the same ensemble of records and the same frequency range, the spectra normalized according to peak acceleration show important site-dependent effects, whereas the spectra normalized according to peak velocity do not.

These results are important, for site-dependent effects are observed only when data are normalized according to peak accelerations and not when other normalization procedures are used. As a result, and in view of the questionable nature of the use of peak accelerations to normalize spectra over a wide frequency range, it is recommended that prior site-dependent results based on this practice be reevaluated.

CHAPTER 5

EFFECTS OF INCLINED ROCK STRATA ON SITE RESPONSE

5.1 ANALYSIS TECHNIQUES

Finite element procedures can be used to carry out site response analyses at nuclear plant sites, in order to assess potential differences in motions at the ground surface and foundation levels. Two example rock sites are considered here, one with horizontally layered limestone and siltstone strata, and the second with these strata inclined rather than horizontal. In this way the effects of strata inclination on the site response characteristics are demonstrated.

5.1.1 BACKGROUND

The Seismic and Geologic Siting Criteria for nuclear plants contained in Appendix A, 10 CFR, Part 100 (USAEC, 1973a) sets forth requirements and procedures for determining vibratory ground motions induced at nuclear plant sites by the Safe Shutdown Earthquake (SSE) and the Operating Basis Earthquake (OBE). Paragraph V-a-1 of this document states:

The maximum vibratory accelerations of the Safe Shutdown Earthquake at each of the various foundation locations of the nuclear 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....

Once the maximum vibratory accelerations are specified according to this document, it remains to develop the design time histories and response spectra for the nuclear plant site. Accepted practice for carrying this out is described in Section 3.7.2 of the USNRC Standard Review Plan (USNRC, 1975), and is summarized as:

- a. Develop design response spectra for the ground surface using the provisions of Regulatory Guide (RG) 1.60 (USAEC, 1973b). RG 1.60 provides standard spectral amplification factors, but also notes that there may be circumstances where site-dependent finite element analyses may be more appropriate for developing ground surface criteria spectra.
- b. Using time histories developed from the above design response spectra, together with an appropriate model of the subsurface site conditions, compute corresponding free-field motions at the embedded foundation levels of the Category 1 structures and at the base of the soil deposit. The motions at the structure foundation levels should have spectral amplitudes that are at least 60% of the design response spectral amplitudes, unless lower values are justified on a case-by-case basis.

These provisions of Appendix A, 10 CFR, Part 100 and the USNRC Standard Review Plan emphasize the need for considering subsurface site conditions when developing vibratory motion criteria. They attach particular importance to estimating the degree to which earthquake motions at the structure foundation levels might differ from those at the ground surface of the nuclear plant site. Site response analysis techniques, based on the dynamic properties of the underlying soil and rock materials, are required to carry out these estimates.

Both one-dimensional and two-dimensional analysis techniques have been used to estimate

site response characteristics. One-dimensional techniques are most widely used in current practice, and are capable of computing horizontal motions due to vertically propagating shear waves at sites with horizontal layers. However, such techniques are not valid for rock sites or soil sites with inclined layers and do not provide a means for estimating vertical motions (SW/AA, 1975).

A more general approach for performing site response analyses is provided by two-dimensional finite element procedures. Such procedures are not restricted to consideration of horizontal motions in horizontally layered sites – rather they can analyze both horizontal and vertical motions at sites with arbitrary horizontal and inclined layers. Finite element techniques have been used in the past to analyze the earthquake-induced response of soil deposits underlain by sloping rock surfaces (Idriss and Seed, 1967), earth banks (Dezfulian and Seed, 1970), and earth-fill dams (Dibaj and Penzien, 1969).

5.1.2 EXAMPLE SITE PROFILES

Example site profiles are shown in Figure 5-1. The site with horizontal rock strata consists of a weathered rock zone underlain by units of limestone, siltstone, and a dolomite formation. The second site also has an upper zone of weathered rock; however, underlying limestone and siltstone units dip at an angle of about 45 deg. The weathered rock zone for both sites is 30 ft thick throughout. The base of the nuclear plant island foundation is assumed to be 94 ft below the ground surface.

The properties of the various rock strata at the example sites are given in Table 5-1, which indicates that, except for the weathered rock, the various rock units have P-wave and S-wave velocities that are within 20% of one another. However, the weathered rock has

wave velocities that are lower than those of the underlying units by factors of about 2 or more.

5.1.3 ANALYSIS PROCEDURE

To evaluate the earthquake-induced response characteristics of the example sites, a total of three two-dimensional finite element analyses were carried out. The first two analyses (Cases 1 and 2) were based on the horizontally layered site shown in Figure 5-1a, whereas the third analysis (Case 3) considered the site with inclined rock units (Fig. 5-1b). The only difference between the Case 1 and Case 2 analyses was the boundary condition at the base of the finite element grid: the Case 1 analysis considered this boundary to be free, whereas the Case 2 calculations utilized special energy-absorbing dashpots along this boundary. The purpose of these two comparative analyses was to assess the feasibility of using the free vs. energy-absorbing boundary along the base of the grid. Results of this assessment (see App. A, AA, 1976a) were used as a basis for defining the base boundary condition for the Case 3 analysis.

The main parameter used in the various site response analyses was the spectral amplification function (SAF), a frequency-dependent quantity defined as

$$\text{SAF}(\omega, x, y) = \frac{S_v(\omega, x, y)}{S_{v_{in}}(\omega)} \quad \omega = \omega_1, \dots, \omega_n \quad (5-1)$$

where

$$\begin{aligned} S_v(\omega, x, y) &= \text{Response spectrum amplitude at frequency } \omega, \text{ computed at given position in grid defined by coordinates } x, y \\ S_{v_{in}}(\omega) &= \text{Response spectrum amplitude at frequency } \omega, \text{ from incident input motions to analysis} \\ \omega_1, \omega_n &= \text{Initial and final frequency values considered in computation of SAF} \end{aligned}$$

(a) Site with horizontal rock strata

(b) Site with inclined rock strata

FIGURE 5-1. EXAMPLE SITE PROFILES

TABLE 5-1. PROPERTIES OF SUBSURFACE MATERIALS CONSIDERED IN EXAMPLE SITE RESPONSE ANALYSES

Material	Modulus of Elasticity, psi	Poisson's Ratio	Shear Modulus, psi	Unit Weight, pcf	P-Wave Velocity, fps	S-Wave Velocity, fps
Weathered Rock Zone	250,000	0.3	96,000	155	3,200	1,700
Unit A Upper Siltstone	1,000,000	0.3	385,000	165	6,200	3,300
Unit A Limestone	1,500,000	0.3	577,000	165	7,600	4,000
Unit A Lower Siltstone	1,000,000	0.3	385,000	165	6,200	3,300
Dolomite Formation	1,500,000	0.3	577,000	165	7,600	4,000
Unit B Limestone	1,300,000	0.3	500,000	160	7,300	3,900

The SAF has been used in prior site response analyses, such as those of Roesset and Whitman (1969). The main advantage of this parameter is its simplicity and its ability to define frequency-dependent site response characteristics in a manner that is not strongly dependent on either the particular input motions used in the analysis or on the damping ratio used in determining the response spectra. Also, the SAF can be readily determined at any selected node point and therefore can serve to indicate how the site response characteristics vary with depth or with horizontal position within the grid.

5.1.4 USE OF TRI/SAC CODE

The TRI/SAC code, which was used to perform all analyses here, is a large capacity computer program for one-, two-, or three-dimensional analyses of structural or continuum systems using the finite element approach (AA, 1976c). TRI/SAC utilizes the Rayleigh damping model. In this, the material damping of the system is assumed to have the following form:

$$[C] = \alpha [M] + \beta [K] \quad (5-2)$$

where $[C]$, $[M]$, and $[K]$ are damping, mass, and stiffness matrices, respectively, and α and β are the damping constants. The choice of damping constants α and β depends on the type of problem being solved. In general, the critical damping ratio for each mode λ_n is related to α and β through the following relationship:

$$\lambda_n = \frac{\alpha}{2\omega_n} + \frac{\beta}{2} \omega_n \quad (5-3)$$

where λ_n and ω_n are the damping ratio and frequency of the n th mode. Equation 5-3 indicates that the Rayleigh damping constants α and β can be selected to represent specified damping ratios for two normal modes, provided the frequencies of those modes are known; the corresponding damping ratios in the remaining

modes of the system are determined by substituting α and β into Equation 5-3.

Another feature of the TRI/SAC code is its energy-absorbing boundary condition. This feature is important for the analysis of continuum systems that extend infinitely in one or more directions. Since the finite element representation of such a system must have finite dimensions, the resulting truncation of space creates an artificial boundary from which reflected signals could distort the true response of the system. Therefore, to minimize these unwanted reflections, an energy-absorbing boundary condition originally formulated by Lysmer and Kuhlemeyer (1969) has been incorporated into TRI/SAC. This consists of a set of orthogonal dashpots attached to each boundary node point and having dashpot constants based on the properties of the continuum at that location.

5.1.5 DEVELOPMENT OF SITE MODELS

5.1.5.1 Finite Element Grid and Material Properties

The finite element grids used in the analyses of the two example site profiles are shown in Figures 5-2 and 5-3. Both grids extend to a depth of 774 ft and have a horizontal dimension of 1050 ft. The grids have free boundaries along the top and left sides, and energy-absorbing boundaries along the right side. Horizontal and vertical input motions were applied to the grid along its left side boundary. The bases of the grids for Cases 1 and 3 have free boundaries, whereas for Case 2 the base of the grid has energy-absorbing dashpots.

The basic element sizes used in the finite element models were dictated by cost considerations and by the requirement that the elements

FIGURE 5-2. FINITE ELEMENT MODEL FOR SITE CONFIGURATION WITH HORIZONTAL ROCK UNITS (CASES 1 AND 2)

FIGURE 5-3. FINITE ELEMENT MODEL FOR SITE CONFIGURATION WITH INCLINED ROCK UNITS (CASE 3)

should not distort seismic waves with frequencies up to some specified cut-off value. Experience with TRI/SAC has indicated that cut-off frequency $f_{\max}$ can be estimated in terms of the shear wave velocity V_s of each material in the grid and the maximum element size d for that material according to the following expression:

$$f_{\max} \leq \frac{V_s}{4d} \quad (5-4)$$

When using Equation 5-4 to estimate element sizes, it was judged that, since the predominant frequencies for most nuclear plants are in the 2 Hz to 10 Hz range, $f_{\max}$ for the rock elements in the vicinity of the structure foundation should be at least 10 Hz; for those elements far from the foundation, it is generally permissible to have an $f_{\max}$ that is somewhat below this 10 Hz level. With this as a basis, the element sizes were then made as small as possible consistent with cost constraints. Maximum element sizes were thereby selected for each material in the grid (see Table 5-2). Table 5-2 indicates that seismic waves with frequencies ≤ 12 Hz will be transmitted without distortion through rock units in the vicinity of the structure. Only for Unit A lower siltstone elements in the lower portion of the Case 3 grid does the cut-off frequency fall below 10 Hz;

however, this will not affect the site response in the upper portion of the grid to any significant degree.

The elastic material properties used to represent the various rock units in the site-response analyses correspond identically to those values provided in Table 5-1. In addition to these properties, the Rayleigh damping parameters for the site were selected to give 5% damping at frequencies of 1 and 10 Hz. This defines the damping ratios of other frequencies that fall below 5% of critical within the 1 to 10 Hz range and increase to values above 5% at frequencies beyond this range. Therefore, the Rayleigh damping parameters selected for these analyses provide reasonable damping ratios at predominant frequencies typical of most nuclear plants (about 2 to 10 Hz); beyond this frequency range, however, some overdamping of the computed site responses might be expected.

5.1.5.2 Integration Time Step

The selection of an integration time step for a finite element analysis is based on four considerations: (1) numerical stability, (2) minimum rise time of the input motions, (3) seismic waves with frequencies up to some

TABLE 5-2. ELEMENT SIZES AND CUT-OFF FREQUENCIES FOR UNDISTORTED TRANSMITTAL OF SEISMIC WAVES

Material	Shear-Wave Velocity, fps	Case Number	Maximum Element Dimension, ft	Cut-Off Frequency, Hz
Weathered Rock	1,700	All cases	35	12
Unit A Upper Siltstone	3,300	1 and 2	35	24
		3	60	14
Unit A Limestone	4,000	1 and 2	35	29
		3 (upper region of grid)	40	25
		3 (lower region of grid)	95	11
Unit A Lower Siltstone	3,300	1 and 2	60	14
		3 (upper region of grid)	40	20
		3 (lower region of grid)	100	8
Dolomite Formation	4,000	1 and 2 only	100	10
Unit B Limestone	3,900	3 only	35	28

specified cut-off value should not be distorted, and (4) cost constraints. Item 1 above is not a problem since a stable numerical integration method by Wilson-Farhoomand-Bathe (1973) is incorporated into TRI/SAC. Regarding Item 3, experience with TRI/SAC has indicated that the cut-off frequency $f_{\max}$ can be expressed in terms of the integration time step Δt as

$$f_{\max} < \frac{1}{4\Delta t} \quad (5-5)$$

An integration time step of 0.01 sec was used in the site-response analyses to provide a good definition of the input motion history (Item 2) and reasonable computer costs (Item 4). Equation 5-5 indicates that this time step will not distort seismic waves with frequencies up to 25 Hz, which is near the upper-bound of the cut-off frequencies associated with the element sizes that are indicated in Table 5-2 (Item 3).

5.1.5.3 Input Motions

The vibratory motion criteria for the two example sites were represented by the Regulatory Guide (RG) 1.60 spectrum shapes scaled to correspond to a peak horizontal acceleration of 0.18 g and a peak vertical acceleration of 0.12 g (Fig. 5-4). The selection of input motion histories to match these criteria spectra was based on the following considerations: (1) the motions should consist of accelerograms measured at sites similar to the example sites, (2) the spectra from the input motions should be similar to those from the assumed design criteria, and (3) the duration of the input records should remain within the computer budget.

The input motions for the site-response analyses were selected to correspond to the first 3 sec of the N69°W and vertical components of the Castaic records measured during the 1971 San Fernando earthquake. These ground motion records were scaled to provide

(a) Horizontal spectra

(b) Vertical spectra

FIGURE 5-4. COMPARISON OF INPUT RESPONSE SPECTRA WITH CRITERIA SPECTRA FOR EXAMPLE SITES

close correlation with the design spectra, and then used as horizontal and vertical input motions applied simultaneously to each node point along the left boundary of the finite element grids. The site conditions at the Castaic accelerograph station are very shallow soil deposits overlying rock, hence similar to those of the example sites. The 3 sec duration was selected to reduce computer costs, while preserving the primary dynamic characteristics of the complete Castaic records.

The scale factors applied to the 3 sec Castaic records were 1.00 for the horizontal input and 1.33 for the vertical input. Figure 5-4 indicates that the scaled Castaic spectrum matches the design spectrum above 0.8 Hz and 3.5 Hz for horizontal and vertical motions respectively. Below these frequencies, the scaled spectra fall below the design spectra; however, in the predominant frequency range for most nuclear plants, about 2 to 10 Hz, the agreement is satisfactory. For actual design analyses, longer records that would yield spectra that match the criteria spectra over a much wider frequency band would be required. Also, the input would normally be scaled to yield response from the free-field model that would match the criteria spectra at either the ground surface or the base of the foundation of the structures. While it is not germane to this investigation, the response of the free-field model at a depth of 94 ft below the ground surface closely approximates the input (and criteria) spectra. Significant amplification in response occurs at the ground surface.

5.2 ANALYSIS RESULTS

Two sets of analysis results are presented here, the first corresponding to Case 1 (horizontal rock strata with a free bottom boundary) and the second to Case 3 (inclined rock strata with a free bottom boundary). Analysis results from Case 2 (horizontal rock strata with energy-absorbing dashpots along the bottom boundary) are omitted here (see App. A, AA, 1976a).

Results are provided in terms of spectral amplification functions (SAF), because these functions provide an indication of the site-response characteristics without being strongly sensitive to the nature of the input motions or to the damping ratio of the response spectra.

Also, only the site response in the upper 215 ft is discussed, since this region is of most interest with regard to the earthquake-induced stresses imparted to embedded nuclear plants. Node points for SAF curves are shown in Figures 5-2 and 5-3.

5.2.1 RESULTS FROM CASE 1 ANALYSIS

The first set of results consists of overlain SAF plots at the ground surface and at depths of 30-ft, 94-ft, and 214-ft (Figs. 5-5, 5-6, and 5-7). These SAF plots are presented at three different distances relative to the left edge of the finite element grids: 105 ft, 455 ft, and 805 ft. The second set of results shows the variation of the horizontal and vertical response characteristics as the seismic waves propagate across the grid (Fig. 5-8), which are presented for the ground surface and for a depth of 94 ft (the assumed level of the embedded nuclear plant foundation).

A general feature of the results is that both horizontal and vertical site response characteristics at a given depth vary in a manner dependent on the horizontal position within the grid. This can be attributed, in large part, to the effects of reflections and refractions of the seismic waves by the various rock strata, geometric attenuation, and other physically real factors that influence the amplitude and frequency content of the seismic waves as they propagate across the site. In addition, the computed site response is affected to some extent by the assumed uniformity with depth of the input motions applied along the left boundary of the grid, a condition not consistent with the assumed site profile. Prior experience indicates that input motions of this type will adjust to the physical characteristics of the model in a few hundred feet from the input boundary. Analysis of the results indicates that the response of the model had become

(a) Depth-dependent variation of horizontal site response

(b) Comparison of horizontal site response at ground surface and at embedded foundation level

(c) Depth-dependent variation of vertical site response

(d) Comparison of vertical site response at ground surface and at embedded foundation level

FIGURE 5-5. DEPTH-DEPENDENT SITE-RESPONSE CHARACTERISTICS 105 FT FROM LEFT BOUNDARY OF GRID — CASE 1, HORIZONTAL ROCK STRATA

(a) Depth-dependent variation of horizontal site response

(b) Comparison of horizontal site response at ground surface and at embedded foundation level

(c) Depth-dependent variation of vertical site response

(d) Comparison of vertical site response at ground surface and at embedded foundation level

FIGURE 5-6. DEPTH-DEPENDENT SITE-RESPONSE CHARACTERISTICS 455 FT FROM LEFT BOUNDARY OF GRID — CASE 1 HORIZONTAL ROCK STRATA

(a) Depth-dependent variation of horizontal site response

(b) Comparison of horizontal site response at ground surface and at embedded foundation level

(c) Depth-dependent variation of vertical site response

(d) Comparison of vertical site response at ground surface and at embedded foundation level

FIGURE 5-7. DEPTH-DEPENDENT SITE-RESPONSE CHARACTERISTICS 805 FT FROM LEFT BOUNDARY OF GRID — CASE 1 HORIZONTAL ROCK STRATA

FIGURE 5-8. VARIATION OF SITE-RESPONSE CHARACTERISTICS WITH HORIZONTAL DISTANCE OF PROPAGATION — CASE 1, HORIZONTAL ROCK STRATA

almost stable at the 805-ft monitoring point. For verification, the response at the 805-ft distance can be monitored and the phased response used as input at the left boundary for a second analysis. This procedure can be repeated a second time if necessary. However, stability is ordinarily achieved with the second iteration. For the comparative cases considered in this study, iterative solutions to a completely stable response were not provided, as it was considered desirable to demonstrate the general convergence of the constant input to a phased response.

5.2.1.1 Horizontal Site Response Characteristics

The following trends characterize the horizontal responses of the Case 1 site configuration:

- a. The Case 1 site configuration attenuates the horizontal motions (by about 25% relative to the incident input motions) at frequencies less than about 4 Hz. In this frequency range, the site horizontal SAF curves are not strongly dependent on either depth or location relative to the left boundary of the grid.
- b. In the frequency range of about 4 Hz to 10 Hz, the ground surface motions exhibit amplified responses relative to the incident input motions. This degree of amplification is less than a factor of 2. Also, the extent of this frequency range depends on the horizontal position within the finite element grid; at 150 ft, site amplification is observed over a narrow frequency band (about 8 Hz) whereas this frequency band tends to widen as more peaks evolve with increasing distance from the left edge of the

grid until a stable condition is reached.

- c. For frequencies in the 4 Hz to 10 Hz range, the degree of amplification of the horizontal input motions decreases with increasing depth below the ground surface; at depths below 80 to 90 ft, the horizontal motions in this frequency range are attenuated with respect to the incident input motions. The primary amplification of the incident input motions occurs in the weathered rock stratum that comprises the top 30 ft of the site.
- d. In the 4 to 10 Hz frequency range, the horizontal motions at the ground surface are much greater than those at the structure foundation level assumed for these computations (depth = 94 ft). In fact, these differences exceed the USNRC Standard Review Plan specifications, which state that criteria spectra at the foundation level must have amplitudes not less than 60% of the amplitudes at corresponding frequencies of motion at the ground surface (unless justified on a case-by-case basis). Furthermore, since the response at these frequencies may be important for nuclear plant design, these results indicate that it is overly conservative to assume that horizontal ground surface records represent criteria motions at the subsurface foundation level for these sites.
- e. For frequencies greater than about 10 Hz to 15 Hz, the horizontal site response at all locations observed is attenuated with respect to the input motions. This could be caused, to some extent, by the filtering action of

the element sizes used in the finite element models, particularly in the weathered rock zone. However, this trend is also related to the Rayleigh damping assumed for the computations, which results in relatively high damping ratios at these higher frequencies.

- f. The computed horizontal site response characteristics at the ground surface are more dependent on the distance from the left boundary of the grid than are the subsurface site response characteristics. The ground surface response characteristics at distances of 455 ft and 805 ft are more nearly similar to each other than to the ground surface response characteristics computed 105 ft from the left boundary, indicating that a stable condition is being approached.

5.2.1.2 Vertical Site Response Characteristics

The following trends characterize the vertical responses of the Case 1 site configuration:

- a. For frequencies below about 10 Hz, the SAF vertical response amplitudes for the ground surface and for the 30-ft and 94-ft depths are nearly identical to each other, but exceed the SAF amplitudes for the 214-ft depth by as much as 50%. This trend appears strongest for those motions that occur at larger distances from the left boundary of the grid.
- b. The similarity of the SAF amplitudes for vertical responses at the ground surface and at the embedded foundation level suggests that the use of composite vertical response spectra

based on ground surface records (such as RG 1.60 for example) is reasonable for use in representing vertical input criteria at the embedded foundation level. Furthermore, since the vertical motions at the foundation level are well within 60% of the ground surface motions, they satisfy guidelines set forth in the USNRC Standard Review Plan.

- c. The frequency content and amplitudes of the vertical site response at a given location are affected by the distance from that location to the left boundary of the grid. At a distance of 105 ft, the predominant amplification occurs within a single frequency band centered around 1 Hz and is less than 1.5 in magnitude. With increasing distance from the left boundary, the peak amplitudes of the SAF curves tend to increase in magnitude (to values as much as 2.3) and significant amplitude peaks occur at a number of additional frequencies. Eventually, a stable condition will be reached.
- d. For frequencies greater than 10 Hz, the SAF vertical response amplitudes are greatest at the ground surface and attenuate with increasing depth.

5.2.2 RESULTS FROM CASE 3 ANALYSIS

Results for Case 3 are presented like those for Case 1. Site response characteristics are defined in the form of overlain SAF plots at various depths and locations in the grid relative to the left boundary, using the same locations within the grid as for Case 1. This will facilitate comparisons of the two sets of results. The Case 3 results are presented in Figures 5-9 through 5-12, with trends regarding the

(a) Depth-dependent variation of horizontal site response

(b) Comparison of horizontal site response at ground surface and at embedded foundation level

(c) Depth-dependent variation of vertical site response

(d) Comparison of vertical site response at ground surface and at embedded foundation level

FIGURE 5-9. DEPTH-DEPENDENT SITE-RESPONSE CHARACTERISTICS 105 FT FROM LEFT BOUNDARY OF GRID - CASE 3, INCLINED ROCK STRATA

(a) Depth-dependent variation of horizontal site response

(b) Comparison of horizontal site response at ground surface and at embedded foundation level

(c) Depth-dependent variation of vertical site response

(d) Comparison of vertical site response at ground surface and at embedded foundation level

FIGURE 5-10. DEPTH-DEPENDENT SITE-RESPONSE CHARACTERISTICS 455 FT FROM LEFT BOUNDARY OF GRID - CASE 3, INCLINED ROCK STRATA

(a) Depth-dependent variation of horizontal site response

(b) Comparison of horizontal site response at ground surface and at embedded foundation level

(c) Depth-dependent variation of vertical site response

(d) Comparison of vertical site response at ground surface and at embedded foundation level

FIGURE 5-11. DEPTH-DEPENDENT SITE-RESPONSE CHARACTERISTICS 805 FT FROM LEFT BOUNDARY OF GRID — CASE 3, INCLINED ROCK STRATA

FIGURE 5-12. VARIATION OF SITE-RESPONSE CHARACTERISTICS WITH HORIZONTAL DISTANCE OF PROPAGATION — CASE 3, INCLINED ROCK STRATA

response characteristics of this site configuration quite similar to those of Case 1. Hence, they will not be repeated here.

5.2.3 COMPARISON OF RESULTS FROM CASE 1 AND CASE 3 ANALYSES

To assess the effect of the inclination of the rock strata on the site response characteristics, horizontal and vertical SAF curves were compared at four different depths along three vertical lines within the finite element grids (see Figs. 3-9 to 3-11 of AA, 1976a). The site response characteristics were not sensitive to whether or not the subsurface rock strata at the two example sites were inclined or horizontal. The frequencies at which SAF peak amplitudes occur at corresponding locations were nearly identical for the two analyses, and the amplitudes of these peaks similar or slightly lower in magnitude (by 20% at most) for Case 3 (with the inclined rock strata). This trend can be attributed to the fact that the wave velocities of the subsurface siltstone and limestone units are not sufficiently different to produce significant impedance mismatches that would modify the seismic waves as they propagate through the site (see Table 5-1). Only the weathered rock stratum at the ground surface has properties significantly different from the other strata within the site.

5.3 CONCLUSIONS

For both sites, the vertical motions at the ground surface and at the foundation level were quite similar at all frequencies of interest for nuclear plant design. This means that it is reasonable to use ground surface records to represent vertical criteria motions at the foundation level. However, for horizontal motions, ground surface response in the 4-to-10 Hz range substantially exceeded the foundation

level response at corresponding frequencies. Since the response at these frequencies may be important for nuclear plant design, the first conclusion is that it is overly conservative to assume that horizontal ground surface records can represent criteria motions at the subsurface foundation level for these sites.

The second conclusion is that the site response characteristics are insensitive to the inclination of the underlying siltstone and limestone units, which is due to the fact that these underlying units have wave velocities that are not sufficiently different from one another to result in significant impedance mismatches at their interfaces.

Although the results did not indicate significant effects due to the inclination of the underlying rock strata, such trends are dependent on the differences in properties of these strata. If adjacent rock units have P- and S-waves velocities that are sufficiently different from one another, then the resulting site response characteristics will be sensitive to whether or not these units are inclined. Therefore, it is recommended that further parametric studies of the earthquake response of inclined rock sites be carried out. The recommended studies would utilize two-dimensional finite element calculations to answer the following questions: (1) what kinds of differences between material properties of adjacent rock units are required before the effects of the inclination of this rock unit becomes significant? and (2) for rock units with sufficiently different properties, how does the site response vary as a function of the dip angle? It would also be desirable to iterate one analysis to stable response relative to distance from the input boundary. The Case 1 analysis, for example, could be repeated using the phased response at the 805-ft station as input to the second analysis.

CHAPTER 6

CORRELATION OF GROUND RESPONSE SPECTRA WITH INTENSITY

6.1 INTRODUCTION

In current seismic design procedures for nuclear plants, Modified Mercalli (MM) site intensities are used to represent the ground shaking induced at the site by the design earthquake events, and are defined from detailed assessments of the seismicity, geology, and tectonics of the region (SW/AA, 1975). Empirical intensity/acceleration correlations are then used to express these intensities in terms of peak horizontal accelerations which, in turn, are used to scale standardized spectrum shapes defined in Regulatory Guide (RG) 1.60 (USAEC, 1973b). These scaled spectra represent the seismic input criteria used in the design of the nuclear plant. (See also, App. A 10 CFR, Part 100, USAEC, 1973a; and Secs. 2.5.2 and 3.7.1 of USNRC, 1975).

Because both the standardized spectrum shapes and the peak acceleration scaling procedures used in current practice have shortcomings, it is important to (1) explore alternate procedures for developing intensity-dependent criteria spectra from the complete library of strong motion data, (2) compare results from these procedures with spectra developed from current practice for defining seismic input criteria, and (3) use these results as a basis for providing recommended spectra suitable for application as seismic input criteria for nuclear plants.

The records used here were measured in the western United States, predominantly in California. Therefore, the results from the investigation are strictly applicable only to the West; i.e., these results may differ from the characteristically slower intensity-attenuation rates in the East and Midwest (Nuttli, 1972).

Nevertheless, the seismic designs of major structures in the East and Midwest have, by necessity, been based on measured records from the West. This practice will continue until a sufficiently large ensemble of strong motion records can be obtained from these areas.

The application of intensities for developing design criteria ground motions requires an awareness of the uncertainties inherent in using subjective intensity ratings to represent the dynamic characteristics of ground shaking at a site. These uncertainties arise because the assignment of intensities is dependent on several factors besides the ground motions themselves. For example, the temperament of individuals interviewed following an earthquake may influence their description of ground shaking effects and may therefore affect the intensity rating. Also, structural damage is strongly dependent on the quality of construction, which may vary considerably between regions and even for structures within a region. Finally, there are uncertainties associated with the use of intensity data from 100- to 200-year-old earthquakes in the seismic design of modern structures, since the type and quality of construction changes considerably with time.

These factors indicate that intensities are not a reliable basis for estimating ground motions for seismic design purposes. Despite this, intensity data will continue to be used in future seismic design applications. Therefore, it is important to improve procedures for estimating ground motions once intensities are specified, in order to reduce the additional uncertainties associated with this step in the development of seismic input criteria.

6.2 DATA DESCRIPTION AND ANALYSIS PROCEDURE

The strong motion data used here were developed during a long term data processing program carried out by the California Institute of Technology (CIT). This program was initiated in 1969 with an objective to provide earthquake engineers and scientists with strong motion data that is processed in a uniform manner and as accurately as possible, within the current technology. The program is described in considerable detail in the various CIT reports containing the data (CIT, 1969-1975) and in various papers (e.g., Trifunac, Udawadia, and Brady, 1973; see also Chapter 4 above).

6.2.1 SITE INTENSITIES

The site intensities assigned to each strong motion record were originally obtained from United States Earthquakes, published annually by the Seismic Engineering Branch of the U.S. Geological Survey. They have been tabulated for each record (see App. B, AA, 1978a) and are identical to those used in other statistical studies based on the CIT free-field data (e.g., Trifunac and Brady, 1975; Trifunac, 1977). The intensities are defined according to the Modified Mercalli (MM) scale (see Fig. 6-1).

The grouping of the data according to intensity levels is summarized in Table 6-1. Most of the data are seen to correspond to Intensities V, VI, and VII; only sparse data are available at higher or lower intensities and, in fact, only the Pacoima Dam record (from the 1971 San Fernando earthquake) has been assigned an intensity greater than VIII.

In view of this distribution of the data, statistical analyses were carried out at five different intensity levels: $\leq$ IV, V, VI, VII, and VIII. The intensities greater than about

VI are usually of greatest interest in nuclear plant design. The lack of data at intensities greater than VIII does not permit statistical analyses at these very high intensity levels, so the reader is cautioned regarding the limited data at Intensity VIII when interpreting and using the composite spectra developed for this intensity. Sample size affects the results for each intensity level; a 95% confidence bounds on the spectral means and standard deviations have been used (AA, 1978a).

6.2.2 DATA CLASSIFICATION

The intensity-dependent data are defined in three classifications: the entire set of data, data based on subsurface soil conditions, and data from local geology (see data category descriptions in Table 6-1). From the first category, intensity-dependent spectra can be obtained without considering the local soils and geologic conditions at each accelerograph station site. Use of these data can indicate the extent to which the amplitudes and shapes of spectra with various damping ratios are dependent solely on site intensity, and also assess seismic input criteria derived using Regulatory Guide 1.60 spectrum shapes, which are based on data that represent a variety of different local site conditions (USAEC, 1973b; Newmark, Blume, and Kapur, 1973).

The second classification is based on the premise that the subsurface soil and rock materials within a few hundred feet of the ground surface have a significant influence on the earthquake motion measurements (Seed, 1969; Schnabel et al., 1972). Prior statistical studies (e.g., Seed, Ugas, and Lysmer, 1976) have classified this data according to three general types of site soil conditions:*

*A fourth site classification is defined as soft-to-medium stiff clays and sands; however, none of the sites considered here fell into this category.

ROSSI-FOREL INTENSITY SCALE (1883)		MODIFIED MERCALLI INTENSITY SCALE (1931), WOOD AND NEUMANN	
		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; DELICATELY SUSPENDED OBJECTS MAY SWING.
II	FELT BY A FEW PEOPLE AT REST; RECORDED BY SEVERAL SEISMOGRAPHS.	III	FELT NOTICEABLY INDOORS, BUT NOT ALWAYS RECOGNIZED AS A QUAKE; STANDING AUTOS ROCK SLIGHTLY; VIBRATIONS 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; AUTOS 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; OVERTURNING OF WALLS, MONUMENTS, AND 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.

ROSSI-FOREL AND MODIFIED MERCALLI (mm) INTENSITY SCALES

JAPAN		I	II	III	IV	V	VI	VII	KAWASUMI (1951)				
RUSSIA	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	MEDVEDEV (1953)
GEOFIAN													
UNITED STATES													
MOD. MERCALLI	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	WOOD AND NEUMANN (1931)

COMPARISON OF THREE MAJOR INTENSITY
SCALES (Trifunac and Brady, 1975)

(a) Intensity scales

FIGURE 6-1. SITE INTENSITIES (Continued next page)

(b) MM intensity map from 1971 San Fernando earthquake

FIGURE 6-1. SITE INTENSITIES (Concluded)

- Deep Soil Sites – Soil deposits whose depth to rock exceeds 250 ft
- Stiff Soil Sites – Firm soil deposits whose depth to rock is less than 150 ft
- Rock Sites – Located on rock outcropping

The accelerograph site soil conditions corresponding to each record were classified into one of the above three categories, provided that reliable soils information was available for that site (SW/AA, 1976). Those sites for which no reliable geotechnical information could be obtained were not assigned a site soil classification. On this basis, 130 of the 186 sets of records were classified into the three site categories, and the records associated with these categories for each intensity level are summarized in Table 6-1. Only 15 of the 130 sets of records represent motions recorded at rock sites, 49 were measured at stiff soil sites, and 66 at deep soil sites. Furthermore, data

were available only at Intensities VI and VII, so only at these intensities are statistical analyses of subsurface soil effects carried out.

The third classification considers that the recorded ground motions have been influenced by subsurface materials that extend to depths comparable to the wavelengths of the significant frequencies of the motions, which may be much greater than depths considered in the second category. To define materials at these extensive depths, geologic maps and other site geology data are required (Trifunac and Brady, 1975; see also Wong and Trifunac, 1977). From Trifunac and Brady (1975), general site classifications are:

- Soft alluvium deposits
- Intermediate (or hard sedimentary) rock
- Basement (or crystalline) rock

One of these three classifications has been assigned to the site of each of the 186 record sets, based on an assessment of data from site

TABLE 6-1. SUMMARY OF STATISTICAL ANALYSIS CASES

Category 1: Data classed according to site intensity only (site conditions not considered)

Category 2: Data classed according to subsurface soil conditions for each site intensity level²

Category 3: Data classed according to local geologic conditions for each site intensity level³

Category	Spectral Damping Ratio	Direction of Motion	Site Conditions	Sample Size for Each Site Intensity Level ¹						
				≤IV	V	VI	VII	VIII	X	Total
1	0.0, 0.02, 0.05, 0.10, 0.20	Horizontal	All site conditions	8	68	132	150	12	2	372
		Vertical		4	34	66	75	6	1	186
2	0.05	Horizontal	Rock	2	0	12	14	0	2	30
			Stiff soil	2	22	18	56	0	0	98
			Deep soil	4	22	44	54	8	0	132
			Total							260
		Vertical	Rock	1	0	6	7	0	1	15
			Stiff soil	1	11	9	28	0	0	49
			Deep soil	2	11	22	27	4	0	66
			Total							130
3	0.05	Horizontal	Basement rock	0	4	14	10	0	2	30
			Intermediate rock	4	30	32	42	0	0	108
			Alluvium	4	34	86	98	12	0	234
			Total							372
		Vertical	Basement rock	0	2	7	5	0	1	15
			Intermediate rock	2	15	16	21	0	0	54
			Alluvium	2	17	43	49	6	0	117
			Total							186

Notes: ¹Site intensities defined according to Modified Mercalli Scale (Wood and Neumann (1931) and Fig. 6-1)

²Subsurface soil conditions categorized according to Seed-Ugas-Lysmer (1976) classification

³Local geologic conditions categorized according to Trifunac-Brady (1975) classification

geology descriptions and geologic maps. Again, the records associated with each local geologic category are summarized in Table 6-1. This table indicates that the local geology data exhibit several patterns similar to the subsurface soil classification. For example, there are relatively sparse data for the basement rock sites, and most of the data are associated with

the alluvium sites; this pattern is similar to that indicated by the rock outcrop and deep soil site data. Also, most of the local geologic data are concentrated at Intensities VI and VII, so again only at these intensities are comparisons of composite spectra for each local geologic condition made.

6.2.3 STATISTICAL ANALYSIS PROCEDURE

The statistics of the various ensembles of data formed for each intensity level and within each level are developed from standard statistical analysis procedures. For each parametric case, the pseudovelocities at a specified frequency are treated as random variables. The ensemble mean, standard deviation, and other pertinent information are then derived at each of the 91 frequency points at which the digitized pseudovelocity amplitudes are defined from the CIT data tapes.

Once these quantities are obtained at each frequency, they are used to derive ground response spectra corresponding to specified probabilities of exceedance; these spectra are the final product of the statistical analysis procedure. This step is implemented here by assuming the pseudovelocities at each frequency to be lognormally distributed. This assumption is based on results from a comprehensive evaluation of this and other possible approaches for estimating probability levels (App. A, AA, 1978a).

6.3 INTENSITY-DEPENDENT SPECTRA

The intensity-dependent spectra were developed according to the data in the three classifications discussed in Section 6.2.2.

6.3.1 INTENSITY-DEPENDENT SPECTRA CONSIDERING ENTIRE DATA SET

The first set of results considers the entire set of 372 horizontal records and 186 vertical records as the basis for developing intensity-dependent spectra. These spectra are developed for site intensities of $\leq$ IV, V, VI, VII, and VIII on the Modified Mercalli scale.

Mean and 84th percentile horizontal and vertical spectra are presented in Figures 6-2 through 6-4 for damping ratios of 0.0, 0.05, and 0.20 (spectra for 0.02 and 0.10 are also

included in AA, 1978a). These figures indicate a generally consistent tendency for the spectra to increase with increasing intensity. However, some exceptions exist with regard to the horizontal and vertical spectra for Intensities $\leq$ IV and VIII. The lower frequency spectral amplitudes for Intensity $\leq$ IV are nearly the same as, or in some cases greater than, the corresponding spectral amplitudes for Intensity V. Similarly, the spectral amplitudes for Intensity VIII are less than or only marginally greater than the amplitudes for Intensity VII over much of the frequency range shown. These exceptions are related to the lack of data at Intensities $\leq$ IV and VIII.

Another trend is that the shapes of the spectra are not the same. The differences in shapes of the various intensity-dependent spectra are most prominent in the frequency range of about 0.2 Hz to 10 Hz, where that portion of the spectra shows a markedly increasing slope with increasing intensity. This trend holds for horizontal and vertical motions and for all of the damping ratios at which the spectra were computed. It suggests that the current design practice, which involves the development of criteria spectra by scaling a spectrum shape that is the same regardless of intensity, may not be adequate.

The effects of data scatter on these trends are indicated by comparing the scatter bands corresponding to the 68% confidence interval (bounded by the 84th percentile and 16th percentile spectra). These effects are shown in Figures 6-5 and 6-6 for horizontal and vertical spectra, and only for spectra with damping ratios of 0.05, since they are generally representative of the other spectra.

Figures 6-5 and 6-6 show two important tendencies. First, in general the spectra increase in amplitude and change in shape with increasing intensity, except for the confidence

LEGEND	INTENSITY
—	≤ IV
- - -	V
- · - · -	VI
- · - · -	VII
- x x -	VIII

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-2. INTENSITY-DEPENDENT SPECTRA — DAMPING RATIO = 0.0

LEGEND	INTENSITY
—	≤ IV
- - -	V
- · - ·	VI
- · - ·	VII
- x x -	VIII

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-3. INTENSITY-DEPENDENT SPECTRA — DAMPING RATIO = 0.05

LEGEND	INTENSITY
—	≤ IV
- - -	V
- · - · -	VI
- · - · -	VII
- · - · -	VIII

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-4. INTENSITY-DEPENDENT SPECTRA — DAMPING RATIO = 0.20

LEGEND	SITE INTENSITY			
	6-5(a)	6-5(b)	6-5(c)	6-5(d)
≤ IV	VI	VI	VIII
V	V	VII	VII

(a) Comparison between spectra bands for intensities $\leq IV$ and V

(b) Comparison between spectra bands for intensities V and VI

(c) Comparison between spectra bands for intensities VI and VII

(d) Comparison between spectra bands for intensities VII and VIII

FIGURE 6-5. 68-PERCENT SCATTER BANDS FOR HORIZONTAL GROUND RESPONSE SPECTRA

LEGEND	SITE INTENSITY			
	6-6(a)	6-6(b)	6-6(c)	6-6(d)
≤ IV	VI	VI	VIII
V	V	VII	VII

(a) Comparison between spectra bands for intensities $\leq$ IV and V

(b) Comparison between spectra bands for intensities V and VI

(c) Comparison between spectra bands for intensities VI and VII

(d) Comparison between spectra bands for intensities VII and VIII

FIGURE 6-6. 68-PERCENT SCATTER BANDS FOR VERTICAL GROUND RESPONSE SPECTRA

intervals for $\leq$ IV and VIII where this trend is weak or nonexistent. Second, the 68% scatter bands are quite wide, with differences of as much as an order of magnitude between upper bound and lower bound spectral amplitudes at some frequencies; furthermore, the confidence intervals for successive intensities often exhibit a noticeable degree of overlap. These facets of the results can be attributed, in large part, to the fact that intensities are an uncertain basis for scaling response spectra or, for that matter, any other quantitative measure of the ground shaking. The many physical parameters related to the geology and tectonics of the region that have a significant influence on ground shaking are not yet well understood, yet their influence is readily apparent in the scatter and overlap indicated in Figures 6-5 and 6-6. Hence, statistical analyses are beneficial since they provide a means of accounting for such uncertainties in seismic design applications.

6.3.2 INTENSITY-DEPENDENT SPECTRA CONSIDERING SUBSURFACE SOIL EFFECTS

Analytical and statistical studies suggest a strong dependence of earthquake ground motions on subsurface soil conditions. These studies indicate that sites with deep soil deposits have larger low-frequency spectral amplitudes and smaller high-frequency spectral amplitudes than do rock sites. Stiff soil sites represent an intermediate condition between deep soil and rock sites (Schnabel, Seed, and Lysmer, 1972; Seed, Ugas, and Lysmer, 1976).

The results presented here are for 5% damped spectra that correspond to a mean statistical level, to an 84th percentile, and to 68% scatter bands. Also, these comparisons of site-dependent spectra are carried out only at Intensities VI and VII, since only at these intensities are data available at each of the

three types of site conditions defined for both the subsurface soil and local geologic classifications. Note, however, that sample sizes are sometimes limited (see Table 6-1) and that limited sample sizes can affect the reliability of the statistical results.

One set of results (Figs. 6-7 and 6-8) provides general trends regarding subsurface soil effects in terms of mean and 84th percentile spectra for Intensities VI and VII. These figures indicate that:

- a. For horizontal motions, the composite spectra for deep soil sites consistently have larger low-frequency spectral amplitudes and smaller high-frequency amplitudes than do the spectra for rock sites. However, the spectra for the stiff soil site conditions do not always represent an intermediate condition between those for deep soil and rock sites.
- b. For vertical spectra, trends pertaining to subsurface soil effects are not always as apparent as for the horizontal spectra. At Intensity VI, the spectral amplitudes for deep soil and rock sites are quite similar at low-to-intermediate frequencies; only at higher frequencies is there a clear distinction between the two sets of spectra. For Intensity VII data, only at low frequencies do the spectral amplitudes of the deep soil and rock sites distinctly diverge according to the trends exhibited by the horizontal spectra; at higher frequencies the spectral amplitudes for rock sites are only slightly greater than for deep soil sites. The vertical spectra for stiff soil sites do not consistently represent a condition that is intermediate between those for deep soil

LEGEND	SUBSURFACE SOIL CONDITON
————	ROCK
- - - - -	STIFF SOIL
- - - - -	DEEP SOIL

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-7. SUBSURFACE SOIL EFFECTS ON GROUND RESPONSE SPECTRA FOR INTENSITY VI

LEGEND	SUBSURFACE SOIL CONDITON
—	ROCK
- - -	STIFF SOIL
- - -	DEEP SOIL

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-8. SUBSURFACE SOIL EFFECTS ON GROUND RESPONSE SPECTRA FOR INTENSITY VII

and rock sites; this trend is similar to that noted above for horizontal motions.

6.3.3 INTENSITY-DEPENDENT SPECTRA CONSIDERING LOCAL GEOLOGIC EFFECTS

Local geologic effects are analogous to the subsurface soil effects. Spectra for deep alluvium sites have larger spectral amplitudes at low frequencies and smaller high-frequency amplitudes than do composite spectra from records measured at basement rock sites (Trifunac, 1977). Spectra corresponding to the intermediate rock site condition should, in theory, fall in between those for alluvium and basement rock.

Mean and 84th percentile spectra developed for three geologic classifications and for Intensities VI and VII are shown in Figures 6-9 and 6-10. These results can indicate whether intensity-dependent spectra follow the trends described above, and are summarized as:

- a. Horizontal spectra for alluvium and basement rock generally follow the expected trends indicated above. Also, as for the subsurface soil comparisons, spectra for the intermediate rock condition do not consistently fall between those for alluvium and basement rock.
- b. Vertical spectra do not always follow the above trends. The spectra for alluvium and basement rock site conditions at Intensity VI are quite similar to low frequencies, whereas at intermediate and high frequencies the spectral amplitudes for alluvium sites fall well below those for basement rock sites. At Intensity VII, the differences between the alluvium and basement rock spectra are relatively

minor at intermediate-to-high frequencies; only at lower frequencies, where the alluvium spectral amplitudes become considerably larger, do the two spectra begin to diverge. For both intensity levels, the vertical spectra for the intermediate rock case do not necessarily fall between the alluvium and basement rock spectra.

- c. The trends indicated above are qualitatively quite similar to those indicated for spectra considering subsurface soil effects.

6.4 COMPARISONS WITH CURRENT PRACTICE FOR DEFINING SEISMIC INPUT CRITERIA

6.4.1 REGULATORY GUIDE 1.60 SPECTRA

Seismic input criteria spectra that correspond to a given site intensity are most often developed by estimating a peak horizontal acceleration that corresponds to this intensity, and then using this acceleration to scale Regulatory Guide (RG) 1.60 spectrum shapes. Although this approach has certain limitations, its wide use suggests the importance of further assessments to see how the RG 1.60 spectrum shapes can be scaled so as to best represent the current library of strong motion records, classed according to site intensity. The intensity-dependent spectra contained in the previous section provide a basis for examining the RG 1.60 spectra in this manner.

Three comparisons between RG 1.60 and the intensity-dependent spectra are presented here. The first considers the RG 1.60 spectra scaled to correspond to a specific site intensity by using various acceleration-intensity correlations that have been widely applied in seismic design practice. The second comparison is based on the RG 1.60 spectra scaled to peak

LEGEND	LOCAL GEOLOGIC CONDITION
————	BASEMENT ROCK
- - - - -	INTERMEDIATE ROCK
- · - · -	ALLUVIUM

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-9. LOCAL GEOLOGIC EFFECTS ON GROUND RESPONSE SPECTRA FOR INTENSITY VI

LEGEND	LOCAL GEOLOGIC CONDITION
————	BASEMENT ROCK
— · — · —	INTERMEDIATE ROCK
- - - - -	ALLUVIUM

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-10. LOCAL GEOLOGIC EFFECTS ON GROUND RESPONSE SPECTRA FOR INTENSITY VII

acceleration levels that represent different probability levels from the data in each intensity grouping. The third comparison considers differences in the shapes of the various intensity-dependent spectra relative to those of RG 1.60, that is, the use of a fixed spectrum shape to represent intensity-dependent spectra whose shapes are actually dependent on the site intensity level.

For consistency with the RG 1.60 spectra, the intensity-dependent spectra used in these comparisons were developed from the data ensemble represented as Category 1 in Table 6-1; i.e., the data are categorized according to intensity but not according to local site conditions. Also, only 5% damped spectra were employed, since they are representative of the others, too.

6.4.1.1 RG 1.60 Spectra Scaled Using Various Acceleration-Intensity Correlations

Several acceleration-intensity correlations have been developed, and typically these are provided in terms of peak horizontal accelerations. Four widely used correlations of this type are compared in Figure 6-11. Corresponding peak vertical acceleration levels are generally taken to be some fraction – usually 2/3 – of the peak horizontal acceleration. Only the Trifunac-Brady study has developed separate correlations between peak vertical acceleration and site intensity.

The RG 1.60 horizontal and vertical spectra are each developed by using the peak horizontal acceleration as a scaling factor. Therefore, for purposes of comparison with the intensity-dependent spectra of the previous section, the horizontal acceleration-intensity correlations shown in Figure 6-11 are used to scale the RG 1.60 horizontal and vertical ground response spectrum shapes. These comparisons are shown in Figures 6-12

through 6-14 for site Intensities V, VI, and VII. Comparisons are not provided at Intensities $\leq$ IV and VIII because of the sparse data at these intensity levels. Figures 6-12 through 6-14 indicate the following trends:

- a. The RG 1.60 horizontal spectra scaled using the Neumann and Trifunac-Brady acceleration-intensity correlations generally fall between the mean and 84th percentile intensity-dependent horizontal spectra. Only in a narrow frequency range (0.2 Hz to 0.6 Hz) for the Intensity V and VI comparisons are the 84th percentile spectra slightly exceeded.
- b. The RG 1.60 vertical spectra scaled using the Neumann and Trifunac-Brady correlations consistently equal or exceed the 84th percentile intensity-dependent vertical spectra.
- c. The RG 1.60 horizontal spectra scaled using the Gutenberg-Richter acceleration-intensity correlation consistently fall between the 16th percentile and mean intensity-dependent horizontal spectra. The RG 1.60 vertical spectra obtained from the Gutenberg-Richter correlation generally fall reasonably close to the mean intensity-dependent vertical spectra.
- d. The RG 1.60 horizontal spectrum scaled using the Hershberger acceleration-intensity correlation falls below the mean intensity-dependent spectrum for Intensity V. At Intensity VI, the RG 1.60 spectrum scaled using Hershberger's correlation is generally similar to the mean spectrum, and at Intensity VII it falls between the mean and 84th percentile spectra for horizontal motions. When vertical spectra are compared, the

FIGURE 6-11. HORIZONTAL ACCELERATION-INTENSITY CORRELATIONS CONSIDERED IN COMPARISONS WITH RG 1.60 SPECTRA

SCALED RG 1.60 SPECTRA	
LEGEND	HORIZONTAL ACCELERATION CORRELATION
————	HERSHBERGER (1956)
-----	GUTENBERG-RICHTER (1942)
-----	NEUMANN (1954)
.....	AVERAGE FROM TRIFUNAC-BRADY (1975a)

(a) Horizontal motions

(b) Vertical motions

FIGURE 6-12. INTENSITY-DEPENDENT SPECTRA VS. RG 1.60 SPECTRA USING VARIOUS ACCELERATION-INTENSITY CORRELATIONS — INTENSITY = V

SCALED RG 1.60 SPECTRA	
LEGEND	HORIZONTAL ACCELERATION CORRELATION
————	HERSHBERGER (1956)
-----	GUTENBERG-RICHTER (1942)
-----	NEUMANN (1954)
.....	AVERAGE FROM TRIFUNAC-BRADY (1975)

(a) Horizontal motions

(b) Vertical motions

FIGURE 6-13. INTENSITY-DEPENDENT SPECTRA VS. RG 1.60 SPECTRA SCALED USING VARIOUS ACCELERATION-INTENSITY CORRELATIONS — INTENSITY = VI

SCALED RG 1.60 SPECTRA	
LEGEND	HORIZONTAL ACCELERATION CORRELATION
————	HERSHBERGER (1956)
-----	GUTENBERG-RICHTER (1942)
-----	NEUMANN (1954)
.....	AVERAGE FROM TRIFUNAC-BRADY (1975)

(a) Horizontal motions

(b) Vertical motions

FIGURE 6-14. INTENSITY-DEPENDENT SPECTRA VS. RG 1.60 SPECTRA SCALED USING VARIOUS ACCELERATION-INTENSITY CORRELATION — INTENSITY = VII

RG 1.60 spectrum scaled using the Hershberger peak horizontal acceleration is similar to the mean vertical spectrum for Intensity V. At higher intensities, it is similar to or greater than the 84th percentile spectrum.

probability levels should be used as the basis for selecting peak accelerations to scale these spectra. To carry out this comparison, peak accelerations associated with the entire group of records from each intensity level are analyzed statistically. Horizontal peak accelerations corresponding to the mean and 84th percentile for each intensity (see Table 6-2) are used to scale the RG 1.60 horizontal and vertical spectrum shapes. These scaled spectra for each intensity level are then compared to the intensity-dependent spectra. Note that the mean acceleration levels correspond closely to

6.4.1.2 RG 1.60 Spectra Scaled Using Peak Accelerations that Represent Various Probability Levels

A second comparison based on RG 1.60 spectrum shapes can provide insight into what

TABLE 6-2. COMPOSITE PEAK ACCELERATIONS FOR VARIOUS INTENSITY LEVELS AND SITE CLASSIFICATIONS

- Category 1: Data classed according to site intensity only (site conditions not considered)
 Category 2: Data classed according to subsurface soil conditions for each site intensity level¹
 Category 3: Data classed according to local geologic conditions for each site intensity level²

Category	Dir. of Motion	Site Conditions	Peak Acceleration (g's) for Each Site Intensity Level ³									
			≤IV		V		VI		VII		VIII	
			Mean	84th p*	Mean	84th p*	Mean	84th p*	Mean	84th p*	Mean	84th p*
1	Hor.	All	0.016	0.022	0.040	0.065	0.085	0.140	0.133	0.190	0.176	0.272
	Vert.	All	†	†	0.021	0.034	0.039	0.065	0.069	0.101	0.134	0.220
2	Hor.	Rock	†	†	†	†	0.110	0.150	0.171	0.238	†	†
		Stiff soil	†	†	0.041	0.061	0.184	0.293	0.122	0.170	†	†
		Deep soil	†	†	0.047	0.076	0.075	0.123	0.136	0.190	0.205	0.316
	Vert.	Rock	†	†	†	†	0.068	0.102	0.092	0.130	†	†
		Stiff soil	†	†	0.027	0.043	0.083	0.131	0.061	0.084	†	†
		Deep soil	†	†	0.019	0.031	0.033	0.055	0.073	0.106	0.110	0.176
3	Hor.	Basement rock	†	†	†	†	0.113	0.152	0.195	0.269	†	†
		Intermed. rock	†	†	0.044	0.068	0.122	0.203	0.135	0.189	†	†
		Alluvium	†	†	0.036	0.058	0.065	0.107	0.130	0.181	0.176	0.272
	Vert.	Basement rock	†	†	†	†	0.072	0.108	0.108	0.131	†	†
		Intermed. rock	†	†	0.025	0.039	0.042	0.069	0.065	0.090	†	†
		Alluvium	†	†	0.018	0.029	0.032	0.053	0.068	0.096	0.134	0.220

*84th percentile value.

†Four or fewer records at this intensity and site classification. Therefore, composite peak accelerations not computed for this case.

Notes: ¹Subsurface soil conditions categorized according to Seed-Ugas-Lysmer (1976) classification.

²Local geologic conditions categorized according to Trifunac-Brady (1975) classification.

³Only peak horizontal accelerations used in scaling of RG 1.60 spectrum shapes.

the Trifunac-Brady and Neumann acceleration-intensity correlations shown in Figure 6-11.

Results from this comparison are shown in Figures 6-15 through 6-17 for site Intensities V, VI, and VII. These comparisons indicate that:

- a. The RG 1.60 horizontal spectrum scaled to the mean peak horizontal acceleration for Intensities V and VI falls above the 84th percentile intensity-dependent spectra at frequencies below 1-to-2 Hz, and falls between the mean and 84th percentile spectra at higher frequencies. At Intensity VII, the RG 1.60 horizontal spectrum scaled in this manner falls between the mean and 84th percentile spectra over the entire frequency range.
- b. When the RG 1.60 horizontal spectrum is scaled to the 84th percentile peak acceleration level, it equals or exceeds the various 84th percentile intensity-dependent spectra over the entire frequency range of interest. The differences between RG 1.60 and the 84th percentile intensity-dependent spectra are greatest at low frequencies for Intensities V and VI; these differences are generally smallest over the entire frequency range for the Intensity VII comparisons.
- c. The RG 1.60 vertical spectra scaled to correspond to the mean and to the 84th percentile horizontal acceleration levels for site Intensities V, VI, and VII exceed the 84th percentile vertical spectra for the corresponding intensity level, often by a substantial amount.

6.4.1.3 Comparisons of Shapes of RG 1.60 and Intensity-Dependent Spectra

The scaled RG 1.60 spectrum shape can

be compared with the shapes of the various intensity-dependent spectra, in order to provide insight into the current practice of using the same spectrum shape to develop criteria spectra, regardless of site intensity.

The comparisons of spectrum shapes for horizontal motions are most readily made from Figures 6-12 to 6-14 since, in these figures, the RG 1.60 and the intensity-dependent spectra for a given probability level have a common point: the peak acceleration (or zero-period spectral acceleration). Therefore, any differences between the RG 1.60 and the intensity-dependent horizontal spectra plotted in these figures correspond to differences in the spectrum shapes. Trends regarding comparisons of the horizontal spectrum shapes are:

- a. For Intensities V and VI, the largest differences in the spectrum shapes occur at frequencies below about 4 Hz where the mean and 84th percentile intensity-dependent spectra are much flatter and have larger amplitudes than the corresponding RG 1.60 spectra. Prominent differences are also noted at higher frequencies.
- b. For Intensity VII, the portion of intensity-dependent spectra at frequencies below about 4 Hz are sloped upward to a much greater degree than for the lower intensities, and therefore more closely resemble the RG 1.60 spectrum shapes in this frequency range. At higher frequencies, the RG 1.60 spectrum shapes have higher amplitudes than do the intensity-dependent spectra. In general, however, the comparisons between these two sets of spectrum shapes are more favorable at this intensity level than for Intensities V and VI.

LEGEND	SPECTRUM
---	RG 1.60 SCALED TO MEAN PEAK HORIZONTAL ACCELERATION
- - -	RG 1.60 SCALED TO 84TH PERCENTILE PEAK HORIZONTAL ACCELERATION
— · —	MEAN INTENSITY-DEPENDENT SPECTRUM
—	84TH PERCENTILE INTENSITY-DEPENDENT SPECTRUM

(a) Horizontal motions

(b) Vertical motions

FIGURE 6-15. INTENSITY-DEPENDENT SPECTRA VS. RG 1.60 SPECTRA SCALED USING VARIOUS PEAK ACCELERATION PROBABILITY LEVELS — INTENSITY = V

LEGEND	SPECTRUM
---	RG 1.60 SCALED TO MEAN PEAK HORIZONTAL ACCELERATION
- - -	RG 1.60 SCALED TO 84TH PERCENTILE PEAK HORIZONTAL ACCELERATION
— · —	MEAN INTENSITY-DEPENDENT SPECTRUM
— · —	84TH PERCENTILE INTENSITY-DEPENDENT SPECTRUM

(a) Horizontal motions

(b) Vertical motions

FIGURE 6-16. INTENSITY-DEPENDENT SPECTRA VS. RG 1.60 SPECTRA SCALED USING VARIOUS PEAK ACCELERATION PROBABILITY LEVELS — INTENSITY = VI

LEGEND	SPECTRUM
---	RG 1.60 SCALED TO MEAN PEAK HORIZONTAL ACCELERATION
---	RG 1.60 SCALED TO 84TH PERCENTILE PEAK HORIZONTAL ACCELERATION
-.-.-	MEAN INTENSITY-DEPENDENT SPECTRUM
—	84TH PERCENTILE INTENSITY-DEPENDENT SPECTRUM

(a) Horizontal motion

(b) Vertical motions

FIGURE 6-17. INTENSITY-DEPENDENT SPECTRA VS. RG 1.60 SPECTRA SCALED USING VARIOUS PEAK ACCELERATION PROBABILITY LEVELS — INTENSITY = VII

For vertical motions, the RG 1.60 spectra and the intensity-dependent spectra plotted in Figures 6-12 to 6-14 do not have a common point as was the case for the horizontal spectra; this is because the RG 1.60 vertical spectra are scaled from the peak horizontal acceleration, rather than from the peak vertical acceleration. Therefore, to establish a common point for comparison, the RG 1.60 and intensity-dependent vertical spectra were scaled to have the same spectral amplitude at 25 Hz.* Comparisons involving the mean and 84th percentile intensity-dependent spectra scaled in this manner are shown in Figure 6-18. These comparisons show the RG 1.60 and the intensity-dependent shapes to have only small differences in amplitude at frequencies above about 6 Hz, and more substantial differences at lower frequencies. Also, the central portions of these two sets of shapes (between about 0.2 Hz and 4 Hz) are most similar for the Intensity V conditions and become increasingly dissimilar with increasing intensity.

6.4.2 VERTICAL VS. 2/3 HORIZONTAL GROUND RESPONSE SPECTRA

The concept of using a standardized spectrum shape that could be scaled to some appropriate ground motion strength level representative of a particular site was first developed by Housner (1959). These spectrum shapes were developed for horizontal motions only, and were based on evaluation of the frequency content of only four sets of records of strong shaking that were available at the time. The corresponding spectra for vertical ground shaking has, for some time, been considered to have the same shape as the

horizontal spectra, and amplitudes that are 2/3 as great (Housner, 1960).

Over the years, considerably more records of strong vertical ground shaking have accumulated, thereby providing a substantially larger vertical motion data base than was available when the above approach for representing vertical motion spectra was first suggested. In fact, this data base is now sufficient to develop standardized vertical response spectrum shapes by processing vertical motion records in the same manner as done for horizontal response spectrum shapes. Despite this, design practice through the years has consistently represented vertical ground response spectra as 2/3 of the horizontal spectra (Newmark and Hall, 1969; Hall, Morhaz, and Newmark, 1976). The only notable exception to this trend has been the RG 1.60 spectrum shapes, in which this 2/3 proportionality factor was not followed over the entire frequency range.

Intensity-dependent spectra can be used to assess this practice for specifying vertical response spectra. This assessment is based on comparisons of composite vertical spectra with 2/3 of the composite horizontal spectra developed for Intensities V, VI, and VII – the intensities with sufficient data for meaningful statistical comparisons. Also, the comparisons are made using the data from Category 1 of Table 6-1, in which data is categorized only according to intensity level. Again, only spectra with a damping ratio of 0.05 are considered.

Results of these comparisons are indicated in Figures 6-19 through 6-21 in terms of mean spectra and 68% scatter bands for each intensity level. These figures indicate that the 2/3 horizontal composite spectra tend to slightly exceed the corresponding vertical composite spectra. Therefore, from a standpoint of conservatism, these comparisons indicate that the representation of vertical design spectra

*For this comparison the actual value is not important and was selected solely to facilitate visual comparison of the spectrum shapes.

(a) Shapes of mean intensity-dependent spectra

(b) Shapes of 84th percentile intensity-dependent spectra

FIGURE 6-18. COMPARISONS OF SHAPES OF INTENSITY-DEPENDENT SPECTRA AND RG 1.60 SPECTRA FOR VERTICAL MOTIONS

(a) Mean spectra

(b) 68-percent scatter bands

FIGURE 6-19. VERTICAL SPECTRA VS. 2/3 X HORIZONTAL SPECTRA — INTENSITY = V

(a) Mean

(b) 68-percent scatter bands

FIGURE 6-20. VERTICAL SPECTRA VS. 2/3 X HORIZONTAL SPECTRA — INTENSITY = VI

(a) Mean

(b) 68-percent scatter bands

FIGURE 6-21. VERTICAL SPECTRA VS. 2/3 X HORIZONTAL SPECTRA — INTENSITY = VII

using 2/3 of the horizontal design spectra is reasonable. However, there are now sufficient data to permit development of composite vertical design spectra from vertical records themselves, rather than from correlations with horizontal response spectra.

6.5 RECOMMENDED SPECTRA FOR SEISMIC DESIGN OF NUCLEAR PLANTS

6.5.1 BASIS AND SCOPE OF RECOMMENDATIONS

The results and comparisons of the two previous sections have important implications regarding the applicability of current procedures for developing seismic input criteria at nuclear plant sites. For example, the shapes of the intensity-dependent spectra were seen to change for different site intensities and were not always well represented by the RG 1.60 spectrum shape. Also, the use of the RG 1.60 spectrum shapes scaled according to a single parameter, the peak horizontal acceleration, was shown to provide inconsistent comparisons with the intensity-dependent spectra developed to correspond to a given probability of exceedance. This latter trend is related not only to the differences in shapes of the RG 1.60 and the intensity-dependent spectra, but also to the uncertainties associated with defining peak accelerations from acceleration-intensity correlations and then using them to scale spectrum shapes over a wide frequency range.

A second result was that, within a given intensity level, spectra grouped according to local site conditions did not indicate consistent or well-defined comparisons. This may be attributed to (1) small sample sizes for several of the intensity-site condition subgroups, (2) uncertainties in defining the site conditions, and (3) other factors that may influence the ground shaking but are not represented by classifying the data solely according to local

site conditions (e.g., source mechanism and source-to-site transmission path).

In view of these considerations, recommendations for representing seismic input criteria motions that correspond to a given site intensity can be given. These recommendations are provided directly in the form of intensity-dependent spectra, and therefore avoid the uncertainties associated with the use of fixed spectrum shapes and peak acceleration scaling. The recommended spectra are based on the results developed from data grouped according to site intensity only, and have the following features:

- a. They are provided for horizontal and vertical motions at Intensities V through VIII. The spectra for Intensities V, VI, and VII are based directly on the results of this study. The spectra for Intensity VIII, an intensity level that is often the basis for developing seismic input criteria at nuclear plant sites in the East and Midwest, are based on an extrapolation of the spectral amplitudes for Intensities V, VI, and VII. This extrapolation procedure was used in place of direct statistical analysis of strong motion data since the available data at this intensity level are so sparse.
- b. The spectra are represented as a series of linear segments that extend between prescribed frequency cutoff points and are fitted to the original spectra. Lognormal mean and 84th percentile spectral amplitudes are defined at these frequency cutoffs for different intensities and damping ratios. The spectra provided in this form can be easily constructed for use in design applications. Also, the

lognormal mean and 84th percentile amplitudes can be used to construct intensity-dependent spectra that correspond to any desired probability of exceedance.

6.5.2 DEVELOPMENT OF BASELINE DATA

The first step in obtaining the recommended spectra is the development of the baseline data. These data come in two parts. First, they consist of intensity-dependent spectral amplitudes over frequencies ranging from 0.06 Hz to 25 Hz for each of the damping ratios. Second, intensity-dependent peak accelerations are defined to provide high-frequency limits for the recommended spectra. Note that these accelerations are not used for scaling purposes, since no normalization of the spectral data is employed.

The baseline spectral amplitudes and peak accelerations for Intensities V, VI, and VII are obtained directly from statistical analyses of the strong motion data at these intensity levels; they correspond to the results previously presented for these intensities in Figures 6-2 to 6-4 and in Table 6-2. However, the strong motion data available for Intensity VIII are too sparse to permit their use in developing baseline results for this intensity level. Therefore an alternate procedure, based on extrapolation of the results from Intensities V, VI, and VII, has been used to define the baseline data for Intensity VIII.

The extrapolation procedure used to develop the Intensity VIII results involves the use of the following exponential regression form that relates the ground motion amplitudes to site intensity:

$$\log_{10} A = aI + b \quad (6-1)$$

where

A = Ground motion amplitude

I = Modified Mercalli intensity

a, b = Coefficients to be determined

The ground motion amplitude, A, represents both the pseudovelocity (ips) at each of 91 frequencies between 0.06 Hz and 25 Hz, and the peak acceleration (g's). In each of these two applications of this equation, the coefficients a and b are defined to provide the best least-squares fit to the mean and 84th percentile ground motion amplitudes for Intensities V, VI, and VII. This resulted in the following expressions for a and b:

$$a = \frac{\sum_i^n I_i y_i - \frac{1}{n} \sum_i^n I_i \sum_i^n y_i}{\sum_i^n I_i^2 - \frac{1}{n} \left[\sum_i^n I_i \right]^2}$$

and

$$b = \frac{1}{n} \sum_i^n y_i - \frac{a}{n} \sum_i^n I_i \quad (6-2)$$

where

I_i = ith intensity value

$y_i = \log_{10} A_i$, where A_i is the ith value of the ground motion amplitude

n = Total number of points fitted by the regression analysis (= 3 for this application, corresponding to motions for Intensities V, VI, and VII).

It is noted that a total of 1824 sets of regression coefficients, a and b, were determined in this application to represent (1) pseudovelocities for two probability levels (mean and 84th percentile), two directions of motion (horizontal and vertical), five damping ratios, and 91 frequencies; and (2) peak accelerations for the same two probability levels and directions of motion. Each of these sets of coefficients was then used in Equation 6-1 with I = 8 to develop the extrapolated ground motion amplitudes for Intensity VIII. The resulting extrapolated spectra and peak acceleration values for Intensity VIII are shown in Figures 6-22 to 6-26

LEGEND	INTENSITY
-----	V
-----	VI
-----	VII
-xx-	VIII

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-22. BASELINE SPECTRA — DAMPING RATIO = 0.0

LEGEND	INTENSITY
----	V
----	VI
----	VII
—XX—	VIII

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-23. BASELINE SPECTRA — DAMPING RATIO = 0.02

LEGEND	INTENSITY
----	V
----	VI
----	VII
xx--	VIII

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-24. BASELINE SPECTRA — DAMPING RATIO = 0.05

LEGEND	INTENSITY
----	V
----	VI
----	VII
XX--	VIII

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-25. BASELINE SPECTRA — DAMPING RATIO = 0.10

LEGEND	INTENSITY
----	V
----	VI
----	VII
xx--	VIII

(a) Mean spectra — horizontal motions

(b) Mean spectra — vertical motions

(c) 84th percentile spectra — horizontal motions

(d) 84th percentile spectra — vertical motions

FIGURE 6-26. BASELINE SPECTRA — DAMPING RATIO = 0.20

and Table 6-3, in which the entire set of baseline results used to develop the recommended spectra are presented.

6.5.3 SPECTRUM CONSTRUCTION PROCEDURE

To develop the recommended spectra, the baseline spectra were fitted with a series of linear segments that intersect at specified frequency cutoff points. Two sets of frequency cutoff points are specified: one for horizontal motions and the other for vertical motions. For a given direction of motion, the same frequency cutoffs are typically used to define the spectra for each damping ratio and probability of exceedance.*

Lognormal mean and 84th percentile pseudovelocities, as obtained from this linear-segment fit, are defined in Tables 6-4 and 6-5 for horizontal and vertical motions respectively. These values are provided for damping ratios of 0.0, 0.02, 0.05, 0.10, and 0.20. To construct the spectra for any of these damping ratios, the appropriate pseudovelocities at each frequency cutoff are simply plotted and connected by straight lines (when plotted on tripartite paper). The construction of spectra at intermediate damping ratios not contained in Tables 6-4 and 6-5 can be carried out using linear interpolation.

For each frequency cutoff point, damping ratio, and direction of motion represented in Tables 6-4 and 6-5, the tabulated lognormal mean and 84th percentile spectral amplitudes can serve to define the spectral amplitude that represents any desired probability of exceedance, p . The following procedure can be

applied for this purpose, in which S_m is defined as the mean value of the spectral amplitude and S_{84} is the 84th percentile value.

- a. Compute the standard deviation of the log of the spectral amplitudes as

$$\sigma = -1 + \sqrt{1 + 2 \log \frac{S_{84}}{S_m}}$$

- b. Using a standard probability table, determine the multiple α such that the mean + $\alpha\sigma$ of normally distributed data corresponds to the probability of exceedance of p percent.
- c. Define the spectral amplitude, S_p , that corresponds to this probability of exceedance from the following expression:

$$S_p = S_{84} \cdot \exp [(\alpha - 1) \sigma]$$

6.6 CONCLUSIONS

Certain features of these recommended spectra can aid in their interpretation and application as seismic input criteria:

- a. The different frequency cutoff points used for the horizontal and vertical motions represent the characteristic differences in the shapes of the spectra that result for these two directions.
- b. The low-frequency ends of the spectra are cut off at a frequency of 0.1 Hz, rather than at the lower frequency limit of the data, which is 0.06 Hz. This is because the spectral amplitudes for the entire range of frequencies below 0.2 Hz and 0.25 Hz (for horizontal and vertical motions, respectively) cannot be represented by a single straight line; instead there is a break in the shapes of the spectra at about 0.1 Hz. Therefore, the proper fitting of these

*The only exceptions occur for the vertical motion spectra with damping ratios of 0.0 and 0.02, where the cutoff frequencies that mark the start of the constant acceleration portion are slightly higher than the corresponding frequencies for the other horizontal and vertical motion spectra.

TABLE 6-3. BASELINE PEAK ACCELERATIONS

Direction of Motion	Intensity	Peak Acceleration, g's	
		Mean Value	84th Percentile Value
Horizontal	V	0 040	0 065
	VI	0 085	0 140
	VII	0 133	0 190
	VIII	0 256	0 352
Vertical	V	0 021	0 034
	VI	0 039	0 065
	VII	0 069	0 101
	VIII	0 125	0 179

Note

- 1 Peak acceleration values for Intensities V, VI, and VII are taken directly from Table 6-2 (Data Category No 1)
- 2 Peak acceleration values for Intensity VIII are derived by extrapolation of the values for Intensities V, VI, and VII, using the regression procedure represented by Equations 6-1 and 6-2

TABLE 6-4 RECOMMENDED SPECTRAL AMPLITUDES FOR HORIZONTAL MOTIONS

Damping Ratio	Frequency,* Hz	Pseudovelocity, ips							
		MMI = V		MMI = VI		MMI = VII		MMI = VIII	
		Mean	84%	Mean	84%	Mean	84%	Mean	84%
0 0	0 10	1 80	2 90	3 00	4 80	5 00	8 20	8 10	13 2
	0 20	3 40	5 80	7 00	12 0	23 0	38 0	55 0	90 0
	3	5 60	9 00	9 00	15 0	17 0	27 0	30 0	46 0
	8	2 10	3 40	3 80	6 00	5 80	9 00	9 20	15 0
	25	0 10	0 16	0 22	0 34	0 34	0 46	0 60	0 90
0 02	0 10	1 60	2 40	2 50	4 0	4 80	7 00	8 00	13 0
	0 20	2 50	4 00	5 50	9 50	18 0	30 0	45 0	72 0
	3	2 70	5 00	5 00	8 00	10 0	16 0	17 0	27 0
	8	0 90	1 40	2 00	3 20	2 70	4 40	4 80	7 00
	25	0 10	0 16	0 22	0 34	0 34	0 46	0 60	0 90
0 05	0 10	1 10	1 80	2 40	3 40	4 3	6 20	7 50	12 5
	0 20	2 00	3 40	4 40	7 40	15 0	26 0	40 0	64 0
	3	2 00	3 30	4 00	6 00	7 80	11 5	13 5	20 5
	8	0 80	1 00	1 40	2 40	2 30	3 20	4 00	6 00
	25	0 10	0 16	0 22	0 34	0 34	0 46	0 60	0 90
0 10	0 10	1 00	1 40	1 80	2 80	3 60	6 00	7 40	12 2
	0 20	1 60	2 80	3 40	5 80	11 50	20 0	32 0	51 0
	3	1 50	2 80	3 00	4 60	5 40	9 00	11 0	16 0
	8	0 60	1 00	1 20	2 20	1 80	2 40	3 60	5 00
	25	0 10	0 16	0 22	0 34	0 34	0 46	0 60	0 90
0 20	0 10	0 70	1 20	1 35	2 20	3 20	5 20	7 00	11 0
	0 20	1 20	1 80	2 40	4 00	8 80	14 0	23 0	36 0
	3	1 10	1 90	2 20	3 70	4 20	6 00	8 20	12 5
	8	0 50	0 70	1 00	1 60	1 45	2 00	2 70	3 80
	25	0 10	0 16	0 22	0 32	0 34	0 46	0 60	0 90

*For cutoff (highest) frequency, see conclusion c in Section 6 6

TABLE 6-5. RECOMMENDED SPECTRAL AMPLITUDES
FOR VERTICAL MOTIONS

Damping Ratio	Frequency,* Hz	Pseudovelocity, ips							
		MMI = V		MMI = VI		MMI = VII		MMI = VIII	
		Mean	84%	Mean	84%	Mean	84%	Mean	84%
0.0	0.10	1.22	2.00	1.34	2.30	1.55	2.50	2.05	2.90
	0.25	2.50	4.00	4.40	8.00	12.0	20.0	29.0	46.0
	4	2.40	3.80	3.50	5.40	7.50	11.0	12.0	18.5
	10	1.20	1.90	1.70	2.70	3.10	5.50	5.00	8.00
	25	0.11	0.18	0.20	0.30	0.33	0.50	0.60	0.95
	40	0.03	0.06	0.06	0.10	0.11	0.16	0.20	0.30
0.02	0.10	1.00	1.70	1.30	1.85	1.50	2.10	1.90	2.90
	0.25	1.70	3.00	3.00	5.00	9.00	16.0	23.0	34.0
	4	1.10	1.90	1.75	3.00	3.60	5.70	6.40	11.0
	10	0.42	0.80	0.80	1.15	1.40	2.00	2.20	3.10
	25	0.07	0.10	0.13	0.20	0.22	0.30	0.39	0.52
	30	0.04	0.07	0.09	0.13	0.15	0.20	0.25	0.35
0.05	0.10	0.82	1.40	1.10	1.80	1.40	2.10	1.90	2.90
	0.25	1.40	2.30	2.30	4.00	6.20	13.0	16.0	28.0
	4	0.70	1.35	1.30	2.20	2.70	4.00	4.60	7.40
	10	0.27	0.56	0.56	1.00	0.85	1.50	1.50	2.20
	25	0.054	0.088	0.10	0.17	0.18	0.30	0.36	0.46
0.10	0.10	0.72	1.14	0.84	1.30	1.25	1.70	1.90	2.60
	0.25	1.00	1.80	1.80	3.00	5.80	9.00	14.0	23.0
	4	0.60	0.92	1.00	1.60	2.00	2.90	3.50	5.50
	10	0.28	0.46	0.44	0.70	0.75	0.95	1.20	1.70
	25	0.054	0.108	0.10	0.17	0.18	0.26	0.36	0.46
0.20	0.10	0.58	0.82	0.70	1.15	1.15	1.65	1.60	2.20
	0.25	0.72	1.20	1.40	2.30	4.10	6.40	10.0	16.0
	4	0.40	0.68	0.70	1.10	1.40	2.20	2.40	3.70
	10	0.20	0.36	0.36	0.60	0.58	0.85	1.00	1.40
	25	0.054	0.088	0.10	0.17	0.18	0.26	0.36	0.46

*For cutoff (highest) frequency, see conclusions c and d in Section 6.6.

low frequency spectral amplitudes would require two linear segments that meet at a frequency of 0.1 Hz. However, because these amplitudes are of no practical importance in nuclear plant design, this additional refinement was judged to be not warranted and the linear-segment fitting of the data was carried out only at frequencies down to 0.1 Hz.

- c. In Tables 6-4 and 6-5, the highest frequency listed for each damping

ratio corresponds to the start of the constant acceleration range of the spectrum. At frequencies equal to or greater than this value, the spectral acceleration corresponds to the peak acceleration value listed in Table 6-3 for the appropriate intensity level and probability. This spectral acceleration value can also be computed as

$$S_a(g's) = \frac{\omega S_v}{386.4}$$

where

$$\omega = 2\pi f_c$$

$$f_c = \text{Cutoff frequency, Hz}$$

$$S_{v_c} = \text{Pseudovelocity (ips) tabulated at frequency } f_c$$

- d. For nearly all of the spectra represented in Tables 6-4 and 6-5, the above cutoff frequency corresponds to 25 Hz; i.e., the constant acceleration portion of most of the spectra defined in these tables starts at a frequency of 25 Hz (which is the maximum frequency at which statistical analyses were carried out for the spectral amplitudes themselves). However, for

two of the spectra, which correspond to vertical motions and damping ratios of 0.0 and 0.02, the constant acceleration portion does not begin until somewhat higher frequencies, as shown in Table 6-5. The cutoff frequencies for these particular cases were obtained by determining the best linear segment fit to the vertical motion baseline spectra between 10 Hz and 25 Hz; this segment was then extended beyond 25 Hz until it intersected with a spectral acceleration equal to the peak vertical acceleration. The frequency at this point of intersection corresponds to the cutoff frequencies given in Table 6-5.

CHAPTER 7

INTENSITY ATTENUATION RELATIONSHIPS

7.1 INTRODUCTION

There are two procedures in general use for predicting the rate of attenuation of the intensity of earthquake vibratory ground motion as a function of distance from the source of a postulated earthquake event. One method relates peak horizontal ground acceleration, earthquake magnitude, and distance. The other relates epicentral Modified Mercalli Intensity (MMI), distance, and attenuated MMI. A conversion relationship between MMI and peak horizontal ground acceleration is then used with the latter.

In western United States, where earthquake strong motion ground accelerations have been recorded, magnitude-distance-acceleration attenuation relationships are generally used. Strong motion accelerograph records provide a direct quantitative measure of the intensity, duration, and frequency content of past earthquake vibratory ground motions. The acceleration record directly provides the maximum ground acceleration at the site. The frequency characteristics of the ground motion are usually obtained from the response spectrum.

One of the important characteristics of the response spectrum is that it converges to the peak recorded ground acceleration at frequencies above about 20 to 30 Hz. Therefore, it is generally assumed that the amplitude of response spectra at all frequencies (for the same damping) are proportional to the peak ground acceleration. (It will be later shown that this is not completely correct.) On this basis, Regulatory Guide 1.60 spectra, which are used for nuclear power plant design, are scaled linearly to the peak ground acceleration predicted for a proposed site (USAEC, 1973b).

For this reason, the peak horizontal ground acceleration, which has been found to be a function of earthquake magnitude and epicentral (or fault) distance, is an important scaling parameter. A typical relationship of these is shown in Figure 7-1. The procedure followed in design is to predict the peak horizontal ground acceleration expected at a site, based on the location and magnitude of the design earthquake and its distance from the site, using relationships such as those given in Figure 7-1. Design spectra, such as Regulatory Guide 1.60 spectra, are then scaled linearly to the predicted site peak horizontal ground acceleration.

FIGURE 7-1. TYPICAL MAGNITUDE-DISTANCE-ACCELERATION-ATTENUATION RELATIONSHIP (Schnabel and Seed, 1973)

Strong motion accelerograph records were first recorded in the United States in 1933 (Long Beach earthquake), but few have been recorded for significant earthquakes. Those recorded in the United States have nearly all been located in California, plus a limited

number in Washington and Montana. Most data on the intensity of ground motion resulting from historical earthquakes in this country must be inferred from isoseismal maps that have been expressed in terms of Rossi-Forel or Modified Mercalli Intensities (MMI). Studies of the attenuation rate in different regions of the United States based on isoseismal maps indicate that attenuation rates in California, for example, are not always the same as in other regions. Therefore, magnitude-distance-acceleration attenuation relationships developed from California earthquake records are considered not applicable to some regions, or at least some adjustment must be made in the attenuation rate. This is usually accomplished by using attenuation data developed from regional isoseismal maps.

Isoseismal maps are prepared for earthquakes in the United States by a cognizant government agency from a damage survey. While an on-site damage survey is made of the epicentral or close-in region, a postcard questionnaire, frequently to postmasters, is usually made for the outlying areas. The isoseismals are then drawn to encompass the different intensity levels. Usually, maps of this type are prepared by individuals with geological or geophysical backgrounds, with the objective of depicting damage levels. In general, isoseismal maps are an indicator of the manner in which the vibratory energy of the earthquake is radiated from the source, but there are factors in the MMI rating system other than the strength of the vibratory ground motion that affect the intensity level assigned.

Modified Mercalli and Rossi-Forel intensity scales do not provide direct quantitative information on the intensity of earthquake ground motion, but are subjective rating systems based primarily upon the observed damage to the ground, earth slopes, buildings, and other

man-made structures following an earthquake. The Mercalli scale as modified in 1931 is currently used, but older records are reported in terms of the Rossi-Forel scale. A brief description of the increments for the two intensity scales is provided in Table 7-1. It is evident in Table 7-1 that ratings by these systems are greatly affected by many factors, which include the quality of construction, the topography, and local subsurface conditions.

Since there is no quantitative information on the rate of attenuation of the intensity of earthquake vibratory ground motion with distance in many regions of the United States (i.e., no strong motion acceleration records), isoseismal maps are frequently used to construct MMI attenuation curves that reflect the average distances from the center of an assumed epicentral region to the various isoseismal lines (e.g., see Fig. 7-2). This MMI attenuation curve is intended to be an indicator of the attenuation rate of the intensity of vibratory motion. For this purpose, it is important that average distances to the isoseismal lines be used, as this will help compensate for localized effects of poor soil conditions, unique structural failures, etc., on the isoseismal map; other adjustments may also be required.

When MMI/distance attenuation curves are used, it is customary to convert the predicted site MMI to an equivalent peak horizontal ground acceleration that is then used to scale design response spectra. An example of the MMI/acceleration relationship proposed by Gutenberg and Richter (1956) is given in Figure 7-2b which, it should be noted, is based primarily on California data. Thus, the initial qualitative isoseismal information on damage, which was prepared by the geologist or geophysicist, is frequently transformed by the design engineer into quantitative design criteria. The procedure is somewhat crude,

TABLE 7-1. COMPARISON OF ROSSI-FOREL AND MODIFIED MERCALLI
INTENSITY SCALES (Thomas et al , 1963)

Rossi-Forel Intensity Scale (1883)	Modified-Mercalli Intensity Scale (Wood and Neumann, 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 noticeably indoors, but not always recognized as a quake, standing autos rock slightly, vibration like 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 awakened, 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, rock-falls, 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 up into air

(a) Modified Mercalli Intensity Attenuation curve for 1971 San Fernando earthquake

(b) Correlation between peak horizontal ground acceleration and Modified Mercalli Intensity

FIGURE 7-2. MODIFIED MERCALLI INTENSITY ATTENUATION RELATIONSHIP

and there is evidence (see Sec. 7-3) that the design engineer has not always correctly interpreted the work of the geologist or geophysicist.

7.2 PEAK GROUND ACCELERATION ATTENUATION RELATIONSHIPS

7.2.1 BEFORE 1971

Three important peak ground acceleration attenuation relationships were developed prior to 1971.

7.2.1.1 Gutenberg and Richter

Gutenberg and Richter (1942, 1956) were among the first to indicate that epicentral peak ground acceleration could be expressed as a function of earthquake magnitude. The first expression was based on 14 earthquakes of which about one-half were of magnitude 5 or less. Epicentral accelerations, however, were not recorded but were extrapolated from other acceleration records for more distantly located stations using strong motion attenuation data developed for the Pasadena laboratory from magnitude prediction studies. The predicted accelerations were associated with the dominant frequency of vibratory ground motion which ranged from about 3 to 5 Hz, depending on the magnitude of the earthquake. Most of the earthquakes had epicenters in the El Centro and Vernon-Long Beach, California, areas, but earthquakes were also included for the Santa Barbara and Bishop, California, areas and for the Hoover Dam, Nevada, area.

In 1956, based on data from a larger number of earthquakes, Gutenberg and Richter revised their first relationship to:

$$\log_{10} A_0 = -2.1 + 0.81M - 0.027M^2 \quad (7-1)$$

where M is the Richter magnitude and A_0 is the peak epicentral horizontal ground acceleration (cm/sec^2). This expression was also based on

epicentral accelerations that had been extrapolated from more distant recording stations, and the accelerations were again associated with dominant frequencies of vibratory motion that ranged from 3 to 5 Hz. The frequency associated with the peak acceleration is important because such accelerations may be associated with higher frequency motion at close distances to the epicenter, which could not have been considered by Gutenberg and Richter.

Equation 7-1 gives epicentral peak ground accelerations for magnitudes 6.5, 7.5, and 8.5 of 0.1, 0.3, and 0.7 g, respectively. These values were indicated to be for rock. For epicenters located in regions where the rock is covered with competent soils, it was indicated that peak ground accelerations should be expected to reach 0.3, 0.6, and 1.0 g, respectively, for these magnitudes; and even higher values were indicated for epicenters located in regions having poor soil conditions. However, the strongest recorded peak ground acceleration in existence at that time was 0.33 g. Earthquake records collected since 1956 do not indicate this degree of magnitude dependence, nor this level of acceleration amplification due to soils (see Sec. 7.2.2). Gutenberg and Richter also provided an attenuation relationship for peak ground acceleration as a function of distance from the epicenter, and a correlation between attenuated MMI and peak horizontal ground acceleration.

7.2.1.2 Housner

Housner (1965) hypothesized that the maximum intensity of ground shaking probably occurs some distance from the fault rather than at the fault surface, and indicated that the peak epicentral ground acceleration anticipated for a magnitude 8.5 earthquake on a deep alluvial firm soil site should not exceed 0.5 g. This was considerably less than predicted by Gutenberg and Richter. The maximum recorded

ground acceleration at that time, as noted above, was 0.33 g, which had been recorded about 4 mi from the surface trace of the May 18, 1940 El Centro earthquake ($M = 6.7$). This site was indicated by Housner to have more than 5000 ft of clay, loam, sand, and gravel overlying sandstone. It probably was the deep alluvial firm soil site to which he referred. Also, similar to Gutenberg and Richter, Housner associated the peak acceleration of 0.5 g to the frequency of the anticipated motion. Housner estimated a dominant frequency of about 2.5 Hz for a magnitude 8.5 earthquake.

Housner identified the intensity of the ground acceleration with the stress drop in the earth's crust resulting from the fault slip, but perhaps incorrectly concluded that the most intense horizontal ground shaking is associated with strike-slip fault displacements, such as associated with the San Andreas fault system. Because of prior movements along this fault system, he postulated that the failure stress was less than the failure strength of the pristine rock. It should be noted that most of the earthquakes that had been considered by Gutenberg-Richter and Housner in the Southern California region up to that time were associated with strike-slip or normal faulting. The much more intense accelerations associated with the 1971 San Fernando earthquake indicate that a more intense stress drop occurs with thrust type faulting.

While Gutenberg and Richter had sought to relate peak ground acceleration with Mercalli Intensity, Housner pointed out that this would never prove practical. He also questioned the logic of using the peak ground acceleration as an indicator of the strength of vibratory ground motion and recommended the use of the spectrum intensity, defined as the area under the velocity spectrum curve between periods of

0.1 to 2.5 sec. While this was a logical recommendation, it never received wide acceptance, probably because of the simplicity and the already established practice of using the peak ground acceleration.

Housner also proposed upper bound relationships between earthquake magnitude and duration of strong ground motion, and between upper bound values of peak ground acceleration, earthquake magnitude, and distance from the fault. The latter relationship (see Fig. 7-3) was almost immediately accepted by the engineering profession and was widely used until after the 1971 San Fernando earthquake. Also shown in Figure 7-3 is the attenuation relationship by Gutenberg and Richter (1956) applied to the peak epicentral ground acceleration of 0.45 g. Part of the difference between the two relationships is explained by the fact that the Gutenberg and Richter relationship does not represent upper bound values. However, the rate of attenuation proposed by Gutenberg and Richter was independent of magnitude and appears to be characteristic of smaller magnitude earthquakes.

For the magnitude, distance, and peak ground acceleration correlation, Housner scaled distances to the point along the fault believed

to represent the center of energy release. For example, although the El Centro recorder was located only 4 mi from the surface manifestation of the fault, a distance of 25 mi was used by Housner as the fault distance, or distance to the center of energy release, as the fault had an estimated length of about 50 mi. In the region very close to the fault, the peak ground accelerations should be a function of the shortest distance to the fault (see Sec. 7.2.3). However, at distances greater than 30 km the Housner assumption is probably acceptable.

Prior to 1971, studies were based on relatively few strong motion records. Gutenberg and Richter in general restricted their data to California earthquakes (which was about the only information available). However, the authors noted that different focal depths and fault conditions might yield different results. Housner lists acceleration, duration, and spectrum intensity data for 15 separate earthquakes, although other records were probably used in the peak ground acceleration-attenuation study. Of the earthquakes listed by Housner, magnitudes varied from 5.0 to 7.6. The Olympia, Washington, earthquake of 1949, which had a focal depth of about 45 mi compared to California depths of 7 to 15 mi, and resulted from normal faulting, was included in Housner's study, as well as the Helena, Montana, earthquake which had a focal depth of about 25 mi. Housner's records included rock sites (Golden Gate) as well as deep soil, stiff soil, and soft clay sites (Ferndale, Taft, and South Pacific Building, San Francisco). Thus, Housner's correlation had variables other than magnitude and distance that were not considered.

FIGURE 7-3. UPPER BOUND MAGNITUDE, DISTANCE ACCELERATION RELATIONSHIPS (Housner, 1965)

7.2.1.3 Seed et al.

Seed et al. (1969) attempted to reduce the effect of variable site soil conditions on the

acceleration-magnitude-distance relationship by developing a relationship for the maximum accelerations on rock. This relationship represents a weighted average of relationships for rock developed previously by Gutenberg and Richter (1956), Benioff (1962), Esteva and Rosenblueth (1963), Kanai (1966), and Blume (1965). The proposed relationships (see Fig. 7-4) were represented as average values, applicable to focal depths of 10 km to 15 km. Except for locations close to the fault, for earthquakes with magnitude 8 or greater, the accelerations were substantially lower than those proposed by Housner (1965), reflecting the amplifying influence of many soil deposits. Again, all epicentral accelerations were based on predicted and not on measured values, and relatively few rock acceleration records were available to any of the investigators, including

Seed. The Seed relationship considered predominant frequencies of motion of 2.5 to 5 Hz, similar to studies by Gutenberg and Richter and by Housner.

7.2.2 AFTER 1971

On February 9, 1971, about 100 ground based strong motion recorders in the Los Angeles area were activated by a magnitude 6.6 earthquake whose epicenter was located near the northern edge of the San Fernando Valley. This single event roughly doubled the available number of strong motion records. Starting in 1972, new peak ground acceleration-attenuation relationships were proposed which in some cases departed radically from those developed prior to 1971. Since these new relationships were influenced by the records compiled in 1971, a brief discussion of the acceleration attenuation data resulting from this one earthquake is given next.

7.2.2.1 Attenuation Relationships for the 1971 San Fernando Earthquake

Locations of the 95 strong motion recorders, as well as the locations of the epicenter and fault of the 1971 earthquake and the resulting isoseismal lines, are shown in Figures 7-5 and 7-6. Identification of the recorder stations, the epicentral distances, station MMI, station subsurface conditions, type of structure housing the strong motion recorders, and the peak horizontal ground accelerations (strong and weak components and the average) recorded for the stations have been tabulated (Tables 2 and 3, AA, 1978b). These data, except for the structures housing the recorders and site soil conditions, are based on data tapes used by Trifunac (1976; see also Trifunac and Lee, 1973). The subsurface conditions used in these tables are divided into the general classifications of rock, and stiff and deep soil sites used by Seed et al. (1976),

FIGURE 7-4. AVERAGE PEAK BEDROCK ACCELERATION MAGNITUDE DISTANCE RELATIONSHIPS (Seed et al., 1969)

FIGURE 7-5. MAP OF STRONG MOTION ACCELEROGRAPH STATIONS RELATIVE TO THE MMI ISOSEISMAL LINES VII AND VIII RESULTING FROM THE SAN FERNANDO EARTHQUAKE OF FEBRUARY 9, 1971

FIGURE 7-6. MAP OF STRONG MOTION ACCELEROGRAPH STATIONS RELATIVE TO THE MMI ISOSEISMAL LINES V, VI AND VII RESULTING FROM THE SAN FERNANDO EARTHQUAKE OF FEBRUARY 9, 1971

rather than the geological classifications used by Trifunac (see also Chapter 6 above).

The peak accelerations recorded for the two components of motion for the 95 stations are plotted in Figure 7-7. Statistically derived best fit curves for the strong and weak components are also indicated in this figure. These curves, which are the statistical means of the two horizontal components, are defined by:

$$\ln A_s = 6.32 - 2.98 \ln D + 0.19 \ln^2 D \quad (7-2)$$

$$\ln A_w = 6.12 - 2.98 \ln D + 0.19 \ln^2 D \quad (7-3)$$

where

A_s = Peak horizontal ground acceleration of the stronger component, g

A_w = Peak horizontal ground acceleration of the weaker components, g

D = Epicentral distance, km
 $20 \text{ km} \leq D \leq 100 \text{ km}$

Several important characteristics of the data in Figure 7-7 are (1) a peak rock acceleration in excess of 1.1 g was recorded at a distance of roughly 9 km from the epicenter; (2) 10 stations were located on rock sites, 37

FIGURE 7-7. REGRESSION ANALYSIS FOR PEAK HORIZONTAL GROUND ACCELERATIONS FOR 1971 SAN FERNANDO EARTHQUAKE DATA (AA, 1978b)

on stiff soil sites, and 37 on deep soil sites (11 unclassified); (3) recorders for most of the soil sites were located in basements of different depths of buildings that were 7 to 30 stories in height, but most of the recorders for the rock sites were mounted on rock surrounded by only a light structure to protect the instrument; (4) similar to previous earthquakes, relatively few records (only one station) were obtained closer than 20 km to the epicenter and for peak accelerations greater than 0.3 g (four stations). Keeping in mind the relatively few rock records, and the possible effect of large buildings and basement depths on the acceleration

records listed for the soil sites, it is noted for later discussion that the correlation study that yielded Equations 7-2 and 7-3 above did not indicate any significant correlatable difference between stiff soil and deep soil sites. A slight correlatable difference was found between rock sites and soil sites. However, many sites are marginal between stiff and deep soil sites, and the subsurface conditions are not reliably known at many other sites. Shifting as few as ten marginal sites from the deep to stiff soil classification gives a correlation that indicates no difference between rock and stiff soil sites but a slightly lower correlation for deep soil

sites. Therefore, the uncertainty in the data does not justify identifying different relationships for different subsurface conditions. Figure 7-8 shows various mean peak horizontal ground acceleration-distance relationships for a magnitude 6.5 earthquake along with the attenuation curves given by Equations 7-2 and 7-3 for the San Fernando 1971 earthquake data.

7.2.2.2 Trifunac

Trifunac (1976) provided attenuation relationships for peak acceleration, velocity, and

displacement as a function of earthquake magnitude, distance, and subsurface conditions (soil, intermediate rock, and basement rock). He also provided relationships for different confidence levels (50 to 90%). However, the study was identified by the author as a preliminary analysis designed to establish approximate epicentral peak values and may not represent peak values that should be used in earthquake engineering design as it does not exclude high frequency accelerations that might occur on rock sites at less than 20 km from the fault. This relationship is plotted in Figure 7-8,

FIGURE 7-8. COMPARISON OF PEAK HORIZONTAL GROUND ACCELERATION-DISTANCE ATTENUATION RELATIONSHIPS FOR MAGNITUDE 6.5 EARTHQUAKES

where it can be seen that for distances between 30 and 100 km – that is, the only range for which there is ample strong motion data – all of the mean correlation curves give comparable results (see Trifunac and Brady, 1975). The Housner curve in Figure 7-8 is an upper bound curve.

Several important assumptions and limitations are noted by Trifunac. For example, the data used were from 187 accelerograph stations (373 horizontal components) representing 57 earthquakes, but more than 50% of the records were from the 1971 San Fernando earthquake. With regard to magnitude, only 3% of the records were for magnitude 4.0 to 4.9, 22% for magnitude 5.0 to 5.9, 71% for magnitude 6.0 to 6.9, and 3% for magnitude 7.0 to 7.9. For the magnitude 6.0 to 6.9 records, 80% were from the 1971 San Fernando earthquake. Also, only 8% of the total sample were for basement rock sites.

The correlations provided by Trifunac were based on a regression analysis using the records noted above and on the following relationships:

$$\log_{10} \begin{Bmatrix} A_{\max} \\ V_{\max} \\ d_{\max} \end{Bmatrix} = M + \log_{10} A_o(R) - \log_{10} \begin{Bmatrix} A_o(M) \\ V_o(M) \\ d_o(M) \end{Bmatrix} \quad (7-4)$$

where

$$\begin{Bmatrix} A_{\max} \\ V_{\max} \\ d_{\max} \end{Bmatrix} = \begin{matrix} \text{Peak ground acceleration,} \\ \text{velocity, and displacement,} \\ \text{respectively} \end{matrix}$$

M = Richter magnitude of the earthquake

$\log_{10} A_o(R_o)$ = An empirically determined function which describes attenuation versus distance

$$\begin{Bmatrix} A_o(M) \\ V_o(M) \\ d_o(M) \end{Bmatrix} = \begin{matrix} \text{Magnitude dependent em-} \\ \text{pirical scaling functions for} \\ \text{acceleration, velocity, and} \\ \text{displacement, respectively.} \end{matrix}$$

The following assumptions for the function $\log_{10} A_o(R)$ are extremely important and are quoted from Trifunac (1976).

[The] equation is based on the assumption that $\log_{10} A_o(R)$ (Richter, 1958) describes approximately the amplitude attenuation with distance, R , for all peaks; i.e., peaks of acceleration, velocity, and displacement. It may seem at first that it is essential to have different attenuation curves for acceleration (high-frequency waves), velocity (intermediate-frequency waves), and displacement (low-frequency waves) peaks, but we found that for the limited number of data points that are available so far $\log_{10} A_o(R)$ may represent a satisfactory first-order approximation for all peaks of strong ground motion (Trifunac and Brady, 1975) for the distance range from about 20 to 200 km.

For the scaling of peak amplitudes presented in this paper, we will neglect the dependence of $\log_{10} A_o(R)$ on wave frequency, recording site conditions, peak amplitudes, earthquake magnitude, and/or on source dimensions and will use the values of $\log_{10} A_o(R)$ which are employed in routine calculations of local magnitudes (Richter, 1958). For large earthquakes which are characterized by long faults and large peak amplitudes, the $\log_{10} A_o(R)$ curve would have a tendency to flatten out for small epicentral distances; while for low magnitude shocks which are typically not accompanied by large source dimensions and large peak amplitudes, this curve would probably have a larger negative slope for small epicentral distances than the curve we propose to use. Such general trends would result from the properties of geometric attenuation with distance and

possibly nonlinear response of soil deposits for large near-field motions. The detailed quantitative description of these effects, however, is beyond the scope of this paper, since the strong-motion data for epicentral distance less than about 20 km, which would be necessary to test the adopted attenuation laws, is completely lacking at this time.

Values for $-\log_{10} A_0$ (R) were compiled by Trifunac (see also Table 4, AA, 1978b), and the average of the 50% confidence level values for $\log_{10} A_0$ (M) for soil and intermediate rock sites given by Trifunac is 1.96. Thus, for peak horizontal ground acceleration and a 6.5 magnitude earthquake, and for soil and intermediate rock sites, Equation 7-4 reduces to

$$\log_{10} A_{\max} = 4.54 - \log_{10} A_0 \text{ (R)} \quad (7-5)$$

This equation was used to construct the Trifunac curve in Figure 7-8. It gives an epicentral value of 1.16 g for a 6.5 magnitude earthquake, which is approximately the value recorded at Pacoima Dam during the 1971 earthquake (1.171 and 1.076 g). This is obviously an excellent correlation with the San Fernando earthquake data. How well future earthquakes with different fault mechanisms will correlate with this expression is open to considerable speculation since the regression analysis has been strongly influenced by the San Fernando earthquake data. Trifunac, it should be noted, cautions against the use of this expression in the eastern United States, and for different fault conditions, indicating that much more data are needed before a reliable relationship can be developed, particularly in the 0 to 20 km range.

The results of the Trifunac study indicate that there was little difference between the peak accelerations recorded on rock sites as compared to those recorded on soil sites. The maximum range in $\log_{10} A_{\max}$ found by Trifunac

between basement rock and soil sites was 0.12. The difference between intermediate rock and soil, however, was 0.06, with the soil sites having the higher values in both cases. These slight differences were essentially confirmed in the development of Equations 7-2 and 7-3, which are based on San Fernando data only. A second conclusion was that the peak ground accelerations are probably not a function of magnitude for magnitudes above about 7.5. While this may be true for epicentral accelerations, fault lengths increase with magnitude, and the total area experiencing a given acceleration level (except for high peaks very close to the fault) should remain a function of magnitude. A final factor is that the Trifunac correlation is not limited to a given range in frequency of motion (i.e., 2.5 to 10 Hz) such as imposed by previous investigators.

7.2.2.3 Schnabel and Seed

Schnabel and Seed (1973) presented an update to the rock acceleration curves by Seed et al. (1969) that is based upon relatively simple wave attenuation assumptions and a correlation with the relatively few acceleration records available for rock sites at that time. Prior to the San Fernando 1971 earthquake, only three or four records were available above a peak acceleration of 0.1 g. Following the San Fernando earthquake, the number of rock records available still did not exceed 15. Therefore, to increase the data sample, rock accelerations were computed for an additional 11 sites using a simple mathematical model based on one-dimensional shear wave assumptions (Schnabel et al., 1972). The study resulted in the attenuation curves given in Figure 7-9; the curve for magnitude 6.6 is shown in Figure 7-8. The Schnabel and Seed curves are based on distance to the fault rather than to the epicenter. However, at distances greater than 20 km, the only distances for which data

are available, the difference between epicentral and fault distance is insignificant considering the scatter in data.

The Schnabel and Seed relationships are primarily theoretical and based on extremely simplified assumptions. The effects of both rock damping and geometric spreading on seismic waves propagating away from the fault were considered. The former required the development of predominant frequencies for the rock seismic wave motion as a function of magnitude and distance. Attenuation due to damping was assumed to be independent of frequency (i.e., the same attenuation per cycle). On this basis, the high frequency motion attenuates more rapidly with distance than lower frequency motion because more cycles occur for the higher frequency motion for a given fault distance. The attenuation due to geometric spreading was expressed as a linear ratio of the area of the wave front at a given distance from the fault to the area of the fault surface. Most of the attenuation with these assumptions results from geometric spreading.

FIGURE 7-9. AVERAGE BEDROCK ACCELERATION MAGNITUDE-DISTANCE RELATIONSHIP (Schnabel and Seed, 1973)

In Figure 7-8, the Schnabel and Seed curve for magnitude 6.6 correlates well with the San Fernando 1971 strong component data for all sites (i.e., rock and soil sites) for distances between 30 and 75 km, and correlates well with the weak component data at distances greater than 75 km. The great difference between peak ground acceleration for soil and rock sites indicated by the earlier Seed et al. (1969) study (compare Housner and Seed curves in Fig. 7-8) is missing from the Schnabel and Seed curve when compared to the San Fernando data. At distances less than 20 km, the difference between epicentral and fault distance becomes important and a direct comparison with Equations 7-2 and 7-3 is not valid. Also, at distances less than 20 km there are few data to corroborate any relationship.

7.2.2.4 Donovan

Donovan (1973b) provided a correlation between peak horizontal ground acceleration, distance from the source, and earthquake magnitude based on 678 earthquake records from North America, Japan, and Papua-New Guinea, and a relationship based on only the 1971 San Fernando earthquake records (214 records, 107 stations). The two relationships (both plotted in Figure 7-8 as a function of epicentral distance) are:

World Data

$$y = 1080 e^{0.5M} (R + 25)^{-1.32} \quad (7-6a)$$

San Fernando Data

$$y = 516,500 (R + 25)^{-2.04} \quad (7-6b)$$

where y is the peak horizontal ground acceleration, cm/sec^2 ; M is the Richter magnitude; and R is the hypocentral distance, the distance to the causative fault, or distance to the center of energy release, if known, in km.

When Equations 7-6a and b are plotted as a function of epicentral distance, Equation 7-6a

converges to a maximum value of about 0.4 g at $R = 0$ and Equation 7-6b to a maximum value of about 0.6 g. Actually, these equations assume R to represent the hypocentral distance, or the distance to the center of energy release, which for California earthquakes are points of reference that are usually 5 to 10 km below the ground surface. Donovan indicates that Equation 7-6b is based on distances to the center of energy release, which for the San Fernando earthquake was estimated to be at a depth of about 5 km. Therefore, the smallest value of R (which occurs near the epicenter) is not zero but 5. On this basis, Equation 7-6b converges to a maximum value of about 0.5 g rather than 0.6 g. In other words, the Donovan curves in Figure 7-8 for distances less than 20 km will be lower than plotted if R represents the distance to the center of energy release, or the hypocentral distance. The distance R may be taken as either the epicentral distance or the hypocentral distance beyond distances of about 20 km with little effect on the resulting peak horizontal ground acceleration. Again it should be noted that at distances less than 20 km there are few data (one station for Eq. 7-6b).

Donovan's work indicates that more intense peak ground accelerations were experienced during the 1971 San Fernando earthquake at distances less than 30 km, than found from the larger data samples that include other earthquakes, keeping in mind the limited data in this range. However, the great difference between the Trifunac and Donovan correlations, and the significant difference between the Seed and Donovan curves for San Fernando data, should not be overlooked. Trifunac's curve in Figure 7-8, for example, converges to a maximum value of about 1.3 g and Seed's to a maximum value of about 0.9 g.

Donovan also lists separate equations for the San Fernando data for rock and soil sites, which indicate great differences in the peak

horizontal ground acceleration in the epicentral region, but approximately the same values at 40 km. Donovan indicates that 56 of the 214 records were on rock, or rocklike sites, which would indicate a completely different site classification than given in tables in AA (1978b). Since there are only two data points for distances less than 20 km (Pacoima Dam), and since correlations by Trifunac do not indicate a significant difference between soil and rock sites, these equations are considered questionable and have not been listed.

7.2.3 SOURCE MODEL STUDIES

Studies indicate that the peak ground acceleration can be associated with the tectonic stress drop at time of faulting (Brune, 1970; Trifunac, 1976; Hanks and Johnson, 1976; and Bernreuter, 1977). It is assumed that the peak ground acceleration is controlled by the stress drop, the distance to the nearest region of the fault that experiences a large stress drop, and the fault dimensions. Stress drop and fault dimensions are used in this relationship in lieu of earthquake magnitude. Bernreuter's expression is:

$$a = \frac{0.071 \Delta\sigma L^{0.77}}{r^{1.66}} \left(\frac{\beta}{\mu} \right) \quad (7-7)$$

where

a = Scaled peak acceleration

$\Delta\sigma$ = Stress drop

L = One-half the length of the highly stressed region of the fault

r = Scaled hypocentral distance

$\left(\frac{\beta}{\mu} \right)$ = Ratio of shear wave velocity to rock density

For the San Fernando earthquake of 1971, Bernreuter used values of 350 bars for $\Delta\sigma$ and 6 km for L to provide a correlation with the records obtained for this earthquake. This equation is shown in Figure 7-10 along with other attenuation relationships for rock sites.

FIGURE 7-10. COMPARISON OF PEAK ACCELERATION ATTENUATION RELATIONSHIPS FOR ROCK SITES

The source model approach is of value when studying data from historical earthquakes where $\Delta\sigma$ and L can be estimated, and peak ground acceleration values are available. However, as a prediction procedure, it suffers the same uncertainty as the other methods, as $\Delta\sigma$ and L are unknown and must be estimated. Bernreuter estimates that $\Delta\sigma$ would normally range between 100 and 1000 bars. The length L to be used for prediction is also uncertain, since, for a given magnitude, variations in L of a factor of 6 are indicated by Greensfelder (1977), based on data from Bonilla (1967). Bernreuter also notes that for large magnitude earthquakes more than one highly stressed region will probably develop along the fault and presently there is no way to estimate how many

highly stressed regions will develop for a given magnitude.

The concepts of the source model can be used to explain why significantly different peak accelerations may result in the very near field (less than 30 km) for earthquakes of approximately the same magnitude, and why (in the very near field) peak accelerations may be practically independent of earthquake magnitude. Source model studies of historical earthquakes are needed to establish the approximate $\Delta\sigma$ and L values that apply to the characteristic faulting of different tectonic provinces. The results of such studies, if successful, might indicate that different very near field attenuation relationships should be applied to different

tectonic provinces. For example, thrust faulting (the 1971 San Fernando earthquake is an example) predominates in the Transverse Ranges province of Southern California; normal faulting is characteristic of the Basin and Range province of Nevada, southeastern California, southern Arizona, and southeastern Idaho; and strike-slip faulting occurs along the northern and southern segments of the San Andreas fault system. Usually greater stress drops are associated with thrust faulting (earth's crust is in compression) than with normal faulting (earth's crust in tension). Strike-slip faulting should have intermediate levels of stress drop. Therefore, at distances less than 30 km, higher peak ground accelerations should probably be associated with thrust faulting than with normal faulting, assuming depth of faulting and related factors are similar.

If this hypothesis could be confirmed, then it would be possible to demonstrate that a Trifunac or Schnabel and Seed peak acceleration attenuation relationship is more applicable to thrust faulting at distances less than 30 km, and that a lower Donovan type attenuation relationship may be more characteristic of strike-slip or normal faulting. At present, however, it cannot be stated whether the San Fernando (1971) acceleration attenuation pattern was anomalously high, or whether it is typical of thrust faulting. So, more earthquakes need to be studied and more acceleration records recorded.

At distances greater than about 30 km from the fault, the type of faulting may not be a factor, for the product $(\Delta\sigma) L$ should be nearly the same for the same energy release. Therefore, for the same magnitude (i.e., the same energy release), greater fault lengths should be associated with normal faults and strike-slip faults than with thrust faults and at source distances equal to or greater than the

fault length, the attenuation should begin to approximate that of a point source.

7.2.4 RESPONSE SPECTRA CONSIDERATIONS

Free-field design response spectra for nuclear power plants are usually developed by scaling Regulatory Guide (RG) 1.60 response spectra linearly to the peak horizontal ground acceleration predicted for the site. Before peak horizontal ground acceleration attenuation relationships can be evaluated, three considerations are (1) whether response spectra can be scaled throughout the entire frequency range as a linear function of the peak horizontal ground acceleration, (2) whether RG 1.60 response spectra can be scaled by peak horizontal ground accelerations that have a different probability of exceedance than the response spectra, and (3) whether these conventional spectra scaling procedures, which are based on acceleration records recorded at distances greater than 20 km from the epicenter, can be used to predict design response spectra for sites that have source distances closer than 20 km.

7.2.4.1 Response Spectra as a Function of Peak Horizontal Ground Acceleration

Free-field horizontal response spectra converge to the peak horizontal ground acceleration at frequencies above about 20 to 30 Hz (see Sec. 7.1 above). Thus, the response acceleration of single-degree-of-freedom (SDF) systems having a fundamental frequency of 20 to 30 Hz, or higher, is the peak horizontal ground acceleration. However, the response acceleration of SDF systems having fundamental frequencies between about 2 and 10 Hz, will generally be 2 to 5 times the peak horizontal ground acceleration (depending upon the level of damping). This results from the fact that the dominant frequencies of the acceleration record usually fall within this range. Response

spectra should, therefore, be scalable as a function of peak horizontal ground acceleration for all frequencies above about 2 Hz. This generally covers the frequency range of greatest interest to nuclear power plant design.

Records for a typical earthquake indicate that the velocity record has a lower frequency content than the acceleration record, and that the displacement record has a still lower frequency content. It is generally held that the response of SDF systems having intermediate frequencies (i.e., 0.25 to 2.5 Hz) will be more of a function of the peak horizontal ground velocity than the peak horizontal ground acceleration since the velocity record has frequencies that are more nearly resonant with this frequency range. Similarly, the response of SDF systems with fundamental frequencies below 0.25 Hz are generally considered to be a function of the peak horizontal ground displacement.

The above relationships have been considered in the formulation of RG 1.60 response spectra, and separate scaling relationships using estimated peak horizontal ground acceleration and peak horizontal displacement are used.* However, peak horizontal ground displacement is expressed in RG 1.60 as a linear factor of peak horizontal ground acceleration so, in effect, the spectra are usually scaled throughout the entire frequency range as a linear function of the estimated peak horizontal ground acceleration.

*Peak horizontal ground velocity is not used since the spectra ordinates for frequencies less than 0.25 Hz are controlled by peak horizontal ground displacement and spectra ordinates for frequencies greater than 2.5 Hz are controlled by peak horizontal ground acceleration. A straight line variation in the spectra (on a logarithmic plot) is assumed between these two frequencies (i.e., 0.25 and 2.5 Hz) eliminating the need to use the peak velocity as a scaling parameter.

Shapes of free-field response spectra are a function of the intensity of ground motion (see Chapter 6). This is demonstrated in Figure 7-11a, where mean + one standard deviation ($M + \sigma$) response spectra are compared with RG 1.60 response spectra scaled to the same peak horizontal ground acceleration. The ($M + \sigma$) response spectra have been developed from a great number of records which were classified according to the Modified Mercalli Intensity (MMI) of the recording station sites. The shape of the spectrum for MMI VII is quite similar to the RG 1.60 spectrum, but for lower MMI values the shapes for frequencies below about 3 Hz depart significantly from the shape of the RG 1.60 spectra. This indicates that the shapes of response spectra are not a completely linear function of the peak horizontal ground acceleration.

The ($M + \sigma$) peak horizontal ground acceleration for the MMI VII spectra given in Figure 7-11a is 0.19 g. This value was used to scale the RG 1.60 spectrum compared with the MMI VII ($M + \sigma$) spectrum in Figure 7-11a. It is not possible to reliably demonstrate what the shape comparison would be at higher peak horizontal ground acceleration values, for there are an insufficient number of earthquake records available at higher intensities (i.e., MMI VIII and IX) to provide a statistical comparison. However, it is reasonable to conclude that the shape of RG 1.60 spectra is adequate when scaled by peak horizontal ground accelerations up to the 0.2 to 0.3 g level, based on California earthquake records. At lower peak horizontal ground acceleration levels, RG 1.60 spectra are appropriate at frequencies above 2.5 Hz which is the frequency range of interest to nuclear power plant design. Also, it should be noted that RG 1.60 spectra generally will be strongly conservative at frequencies below 2.5 Hz for ($M + \sigma$) peak horizontal ground accelerations below 0.2 g.

(a) Comparison of Modified Mercalli response spectra with RG 1.60 response spectra scaled to same peak horizontal ground acceleration

(b) Comparison of MMI VII mean and mean plus one standard deviation spectra with RG 1.60 spectra scaled to comparable peak horizontal ground acceleration (Werner and Ts'ao, 1977)

FIGURE 7-11. COMPARISON OF MODIFIED MERCALLI INTENSITY DEPENDENT RESPONSE SPECTRA WITH RG 1.60 RESPONSE SPECTRA

7.2.4.2 Probability Level of Response Spectra

Figure 7-11b provides a comparison of MMI VII mean and $(M + \sigma)$ spectra with RG 1.60 spectra that have been scaled to the mean and $(M + \sigma)$ peak horizontal ground accelerations of the respective MMI VII spectra. RG 1.60 spectra are generally considered to be equivalent to $(M + \sigma)$ spectra, but are based on a smaller sample than considered in deriving the MMI VII spectra plotted in this figure. Since RG 1.60 spectra are approximately $(M + \sigma)$ spectra, better agreement can be observed in Figure 7-11b between the $(M + \sigma)$ MMI VII spectrum and the RG 1.60 spectrum than occurs between the mean MMI VII spectrum and the RG 1.60 spectrum scaled to the same mean peak acceleration. However, for nuclear power plant sites, the peak horizontal ground acceleration selected for the site and used to scale the design spectra has a higher probability of not being exceeded, for example, an annual probability of about 99.99%.* The question that arises, therefore, is whether RG 1.60 spectrum scaled by a peak horizontal ground acceleration value that has a 99.99% annual probability of not being exceeded yields a design spectrum that has a 99.99% annual probability of not being exceeded throughout its entire frequency range. In view of the fact that the MMI VII mean and $(M + \sigma)$ spectra in Figure 7-11b both lie below the RG 1.60 comparative spectra, it appears that the design response spectra will have an annual probability of not being exceeded comparable to the annual probability of the peak horizontal ground acceleration, or higher, when the peak horizontal ground acceleration is in the 0.2 to 0.3 g range.

*In contrast, $(M + \sigma)$ represents a value that has about an 84% probability of not being exceeded.

7.2.4.3 Scaling Relationships for Epicentral Distances Less Than 20 km

It has been noted that very few acceleration records are available at peak horizontal ground accelerations greater than 0.3 g, and at epicentral distances less than 20 km. Therefore, acceleration records used to develop RG 1.60 spectra, and the MMI spectra discussed above include very few records for epicentral distances less than 20 km. Peak horizontal ground acceleration distance attenuation relationships by Gutenberg and Richter, Housner, and Schnabel and Seed have all indicated that their relationships assume predominant acceleration frequencies between about 2 and 10 Hz, and are based on records that have the same limitations. Only Trifunac and Brady have not indicated this frequency restriction, and their peak horizontal ground acceleration attenuation relationship indicates higher accelerations at epicentral distances less than 20 km than the other investigators. The questions to be considered are (a) whether the higher peak accelerations indicated by Trifunac and Brady at epicentral distances less than 20 km are due to higher frequency accelerations, and (b) how should RG 1.60 spectra which are based on records for stations located in general more than 20 km from the epicenter, be scaled to provide design response spectra for nuclear power plant sites located at distances closer than 20 km to the earthquake source.

Schnabel and Seed (1973) have pointed out that high frequency ground motions attenuate more rapidly with distance than lower frequency ground motions. To test this hypothesis, Page et al. (1972) filtered the Pacoima Dam 1971 San Fernando earthquake record to remove all frequencies greater than 8.5 Hz. The resulting peak horizontal ground accelerations were reduced from 1.25 g and 1.22 g to 0.82 g and 0.93 g for the two horizontal components, respectively. (Note: the baseline corrected

records list peak horizontal ground accelerations of 1.17 g and 1.08 g for Pacoima Dam.) Page's average peak horizontal acceleration from the filtered records of 0.87 g has been plotted in Figure 7-8 for an epicentral distance of 9.1 km; the 0.87 g value agrees closely with the Trifunac and Brady mean curve. This indicates that the Pacoima Dam records have peak horizontal ground accelerations within the frequency range of 2 to 8.5 Hz that reach the 0.8 to 0.9 g level and that the Trifunac and Brady mean curve given in Figure 7-8 is representative of ground motion frequencies considered by earlier investigators.

Figure 7-12 compares the response spectrum for the S14°W component of the Pacoima Dam 1971 San Fernando earthquake record with the RG 1.60 spectrum (5% damping) scaled to both 0.87 g and 1.17 g. The former represents peak horizontal ground acceleration at this site for motions that are below 8.5 Hz and the latter represents the peak for those motions that are

greater than 8.5 Hz. It is evident that scaling to the 1.17 g level provides response much above that actually recorded. Scaling to the 0.87 g level, however, provides a spectrum that envelops the Pacoima Dam spectrum except for frequencies above 10 Hz. This would indicate, at least for this one record, that RG 1.60 spectra should be scaled based on the predicted peak horizontal ground accelerations associated with frequencies between about 2 to 10 Hz. However, close in sites on rock should be expected to experience higher frequency accelerations, and the high frequency end of the design response spectra (above 10 Hz) should be adjusted to incorporate the higher response. Deep soil sites, such as characterized by the El Centro 1940 record, apparently do not transmit the higher frequency accelerations. However, the data available in and near the epicentral region are quite limited, and special studies are required to develop spectra for sites having source distances less than 20 km.

FIGURE 7-12. COMPARISON OF PACOIMA DAM SPECTRUM WITH RG 1.60 SPECTRA

7.2.5 CONCLUSIONS

Important peak horizontal ground acceleration-magnitude-distance relationships have been developed in this country since 1942. However, the early investigations were based on meager strong motion accelerograph data and judgmental postulations. Most records were for soil sites located more than 20 km from the source, which in most cases resulted from earthquakes occurring on normal or strike-slip faults. Epicentral accelerations given by Gutenberg and Richter (1942, 1956) were all extrapolated from strong motions recorded at significant distances from the fault and were associated with dominant frequencies of ground motion that ranged from 2 to 5 Hz. Housner and Seed associated peak horizontal ground acceleration-distance-attenuation relationships with dominant frequency ranges for the vibratory motion of about 2 to 10 Hz.

Most studies since 1971 have been dominated by the data collected from the San Fernando 1971 earthquake. More than 100 ground based stations recorded strong motion accelerations for this one event, thereby doubling the total number of available records. Peak horizontal ground acceleration-magnitude-distance relationships developed by Trifunac and Brady and by Schnabel and Seed following the San Fernando 1971 earthquake indicate significantly higher peak horizontal ground accelerations at distances less than 30 km from the fault than relationships proposed by Housner and Seed prior to 1971. At distances less than 30 km, the Trifunac and Brady peak horizontal ground acceleration-distance-attenuation relationship for magnitude 6.5 earthquakes correlates closely with the mean peak accelerations measured during the 1971 San Fernando earthquake since more than 80% of the data were from this earthquake. The Schnabel and Seed relationship gives peak horizontal ground accelerations that are significantly lower than

given by the Trifunac and Brady relationship in this distance range. Both relationships give approximately the same results for source distances greater than 50 km. Relationships developed by Donovan based on world data and on only 1971 San Fernando data give much lower values than indicated by Trifunac and Brady or by Schnabel and Seed at source distances less than 30 km. A correlation of 1971 San Fernando data does not confirm Donovan's relationship based on 1971 San Fernando earthquake data.

The peak horizontal ground acceleration-magnitude-distance relationship by Schnabel and Seed is indicated to be applicable to rock sites. Correlation studies by Trifunac and Brady and that presented here (AA, 1978b) based on 1971 San Fernando data find only a slight correlatable difference in the attenuation relationship for rock and soil sites. Since the mean correlation provided here for all site conditions based on 1971 San Fernando data has good agreement with the Schnabel and Seed relationship for rock sites at source distances greater than 30 km, and since there are few rock site records, the Schnabel and Seed relationship is concluded to be equally applicable to rock or stiff soil sites in this distance range.

When the source distance is less than about 20 km for rock sites, the peak horizontal ground acceleration may be associated with ground motions that have dominant frequencies of motion greater than associated with the ground motion records used to develop RG 1.60 response spectra (i.e., frequencies greater than 2.5 to 10 Hz). Design response spectra for nuclear power plant sites on rock having source distances less than 20 km can be scaled from RG 1.60 response spectra by using the peak horizontal ground acceleration for the site associated with ground motions having dominant frequencies between 2.5 and 10 Hz. However, the response spectra should be adjusted upward

for frequencies above 10 Hz to account for higher peak ground accelerations associated with these higher frequencies. A study of the Pacoima Dam 1971 San Fernando earthquake record (Page et al., 1972) indicates that the peak horizontal ground accelerations predicted by the average soil and intermediate rock site relationships developed by Trifunac and Brady (Fig. 7-8) are associated with dominant frequencies that fall within the prescribed range of 2.5 to 10 Hz. There are indications that deep soil sites (e.g., El Centro), even at short source distances, do not transmit the higher frequency motions.

It is not known whether the peak horizontal ground accelerations recorded during the 1971 San Fernando earthquake in the very near field are anomalously high, or whether they are characteristic of thrust faulting for focal depths typical of California earthquakes. Studies based on more data from well instrumented earthquakes are needed before this question can be answered. Based on present knowledge, peak horizontal ground acceleration-magnitude-distance relationships that give lower values than those indicated by Schnabel and Seed are not recommended for regions of thrust faulting in the western United States where focal depths of 10 to 15 km, or less, are anticipated, accompanied with surface manifestations of faulting. Professional judgment is required to estimate peak horizontal ground accelerations for sites in other regions of the United States and for different fault conditions and focal depths. In regions where earthquakes have significantly different focal depths than typical of California earthquakes, and in regions where MMI attenuation relations are significantly different than experienced in California, the direct applicability of peak acceleration-magnitude-distance relationships based essentially on California earthquakes is questionable. Attenuation rela-

tionships for these regions are discussed in the following section.

7.3 MODIFIED MERCALLI INTENSITY ATTENUATION RELATIONSHIPS

Strong motion acceleration records are available in the United States for only a few western states. For the rest of the country, the only available information on the intensity of ground motion from historical earthquakes is expressed in terms of Rossi-Forel or Modified Mercalli Intensities (MMI). Gutenberg and Richter (1942, 1956) attempted to correlate MMI with peak ground acceleration, but Housner (1965) pointed out that this would never prove practical for engineering design. Nevertheless, in the absence of other information, there is no alternative but to use MMI data to estimate ground motion intensity attenuation relationships.

7.3.1 GENERAL PROCEDURE

In regions where there are no strong motion earthquake acceleration records, the general procedure for developing attenuation relationships for the intensity of ground motion as a function of distance from an estimated epicenter is first to collect several isoseismal maps for historical earthquakes in the region of study. The average distances from the estimated epicenter to each isoseismal line are determined for each isoseismal map, and an MMI attenuation curve is constructed for each earthquake. An average attenuation curve for all earthquakes studied is then constructed. The standard deviation can also be computed if the attenuation rate is to be established probabilistically. An example of attenuation curves developed for seven earthquakes in the southeastern Appalachian seismic zone is shown in Figure 7-13. The average of these curves is then compared with an attenuation curve for the Charleston, South Carolina, earthquake of 1886.

FIGURE 7-13. NORMALIZED INTENSITY ATTENUATION CURVES FOR EARTHQUAKES IN SOUTHERN APPALACHIAN REGION (See Table 1-3 in Chapter 1, page 1-8)

(Note that the mean curve, along with the Charleston curve pulled out from the grouping of curves, is shown on the right side of Figure 7-13.) One study concluded that the mean of the curves for the seven smaller earthquakes could be used for earthquakes having epicentral intensities of VIII or less, but the attenuation curve for the Charleston earthquake could be used to estimate the attenuation rate of future earthquakes in this region having epicentral intensities of IX or greater (see Chapter 1).

Earthquakes in eastern United States are known to have different ground motion intensity attenuation characteristics than earthquakes in California. Hence, the direct transfer of

magnitude-acceleration-distance attenuation relationships developed from California earthquakes to the eastern United States is questionable. MMI attenuation curves for different regions in the United States are shown in Figure 7-14. All represented 7+ magnitude earthquakes. The most rapid rate of ground motion intensity attenuation occurred for the California earthquake and the lowest rate for the Charleston and New Madrid earthquakes. The epicentral intensity of the latter two were adjusted downward to compensate for the adverse soil conditions that existed in the epicentral areas of these two earthquakes.

Modified Mercalli intensity/distance attenuation curves are used to estimate the MMI that can be anticipated at a proposed site based on the distance from the potential earthquake source to the site. The site MMI is then converted to an estimated peak ground acceleration by using an MMI/peak ground acceleration correlation relationship. The overall method is crude, with significant errors being associated with both the MMI/distance attenuation curve and the conversion of the site MMI to an equivalent peak horizontal ground acceleration.

7.3.2 FACTORS AFFECTING THE ASSIGNED MMI

The MMI rating system is a qualitative system based on the observed damage to the ground and to man-made structures during an earthquake. As a result, the assigned MMI is affected by factors other than the intensity of ground motion, and isoseismic maps may require some adjustment if their intended use is to estimate the attenuation rate of ground motion intensity.

Damage to structures is an important criterion for establishing MMI ratings between VII and IX. Therefore, it is of interest to note the difference in the description of structural damage assigned to MMI VII, VIII, and IX.

FIGURE 7-14. MERCALLI INTENSITY/DISTANCE ATTENUATION RELATIONSHIPS FOR 7+ MAGNITUDE EARTHQUAKES

(see Wood and Neumann, 1931, for a complete MMI description; also Table 7-1 above and Chapter 6). For buildings of good design, or specially designed, damage is negligible at VII, slight at VIII, and considerable at IX. For well-built ordinary structures, the damage is slight at VII, considerable at VIII, and great with partial collapse at IX. Obviously, these damage descriptions are qualitative and relative, and investigators will interpret them differently. Also, structure response is being described, not intensity of ground motion, which is an important difference.

The response of a structure is a function of its fundamental frequency and the frequency content of the ground motion. The greatest response occurs when the two frequencies are resonant. The strength and frequency content of the ground motion varies with distance and may also vary with soil conditions. Thus, at one site, one- and two-story structures of relatively high fundamental frequency may be damaged because of strong high frequency ground motion, but at a greater distance these structures may be undamaged. In contrast, multistory buildings of lower fundamental frequency may be damaged at this greater distance because the lower ground motion frequency may be resonant with the building frequency. Although seldom done, damage comparisons should be based on the same type of structures if the objective is to depict attenuation in ground motion intensity rather than structural damage. Otherwise, adjustments in MMI should be made.

The effect of distance on the strength and frequency content of the ground motion was demonstrated during the Kern County, California, earthquake of July 21, 1952. Accelerograph records were obtained for this earthquake at several stations. The Taft station was located about 25 mi from the epicenter and recorded peak horizontal ground accelerations of

0.16 and 0.18 g (see Fig. 7-15). Seventy-five miles to the south, in Los Angeles, peak ground accelerations of 0.04 to 0.06 g were recorded at the Hollywood Storage Building, indicating that the peak ground acceleration had attenuated to only 25 to 30% of those recorded at Taft. Study of the response spectra for the two records, however, indicates that while the response accelerations at 5 Hz and above for the Hollywood Storage records had attenuated to only about one-third of those experienced at Taft, the response accelerations at frequencies below 1 Hz were approximately two-thirds of those experienced at Taft in the same frequency range. This can be explained by the fact that high-frequency ground motion attenuates more rapidly than low-frequency ground motion (see Schnabel and Seed, 1973). Damage to one- and two-story buildings (fundamental frequencies of 5 to 8 Hz) in the Taft area led to the assignment of an MMI of VII. In certain areas of Los Angeles, damage to 12- to 14-story buildings (frequencies ≤ 1 Hz) also led to the assignment of an MMI of VII, but one- and two-story structures were virtually undamaged. While the assigned MMI in the Los Angeles area is a good indicator of damage done, it is not a good indicator of peak ground acceleration in the 5 to 8 Hz range.

The average distance from the epicenter to the MMI VIII isoseismal is greater than the distance from the epicenter to the Taft station, and the average distance from the epicenter to the MMI VII isoseismal is less than the distance from the epicenter to the Los Angeles area (Fig. 7-15). Therefore, as an indicator of strength of ground motion, the Taft station should be considered an MMI VIII, and the Hollywood Storage site should be considered an MMI VI. Unfortunately, correlation studies which relate MMI and peak ground acceleration generally consider only the damage assessment. Alternative procedures are needed.

FIGURE 7-15. ISOSEISMAL MAP FOR KERN COUNTY EARTHQUAKE OF 21 JULY 1952
(Steinbrugge and Moran, 1954)

Topography and subsurface soil conditions also have an important effect on the assigned MMI. This can be illustrated by the following MMI descriptions. At VIII, sand and mud are ejected in small amounts (liquefaction); at IX the ground is cracked conspicuously and underground pipes are broken; at X the ground is badly cracked, rails are bent, landslides occur from river banks and steep slopes, and there is shifted sand and mud; and at XI broad fissures occur in the ground, there are earth slumps and land slips in soft ground, underground pipes are completely out of service, and rails are bent greatly. It should be evident that if the epicentral region is underlain with rock or firm soils with no ground water present, and if the topography is flat, lower MMI classifications should result than will be assigned to an epicentral region having saturated soils, streams, and steeply sloping topography. For example, the epicentral region of the 1952 Kern County earthquake (magnitude 7.5) was classified IX and X, with the latter being a reasonable maximum. The New Madrid, Missouri, earthquake of December 16, 1811 has been estimated by Nuttli (1973) to have had a comparable local magnitude of 7.5 but an MMI of XI was assigned to the epicentral region. Since the epicentral area was located in the flood plain area of the Mississippi River and was practically uninhabited in 1811 (i.e., no structures present), the epicentral intensity was based largely on the drastic changes in topography and water levels that occurred (Nuttli notes that one lake was raised up higher than the surrounding country and drained through fissures in the bottom of the lake, but nearby water rose above the ground surface to waist depth, and there was ground subsidence of as much as 12 ft). If the epicenter had been located in a region of stiff soils, no ground water, and flat topography, it is doubtful whether an epicentral intensity greater than X would have been assigned. Richter (1958), for

example, has indicated the following relationships between magnitude and epicentral intensity for California earthquakes:

$$M = 1 + \frac{2I_0}{3} \quad (7-8)$$

where M is the Richter magnitude and I_0 is the epicentral MMI. This equation gives an I_0 of slightly less than X for a magnitude of 7.5.

Whether an epicentral intensity of X or XI should be assigned becomes important when attenuation curves are constructed for the purpose of estimating the rate of attenuation of vibratory ground motion. Figure 7-16, for example, shows that the attenuation curve for the New Madrid 1811 earthquake correlates better with four other regional earthquakes having firm soil epicentral conditions when an epicentral intensity of X (i.e., firm soil condition) is assumed rather than XI for the 1811 earthquake. Adjustments in the attenuation curve should therefore be made to reflect firm site conditions when the intent is to use the results to depict the attenuation rate of peak ground acceleration.

Failure of unique structures may lead to the assignment of an unnecessarily high MMI. For example, bridge failures are listed as an indicator of MMI XI, and bridge failures in the San Fernando valley were early considered to be a reason for assigning an MMI XI to the epicentral area of this earthquake. Ground fissures, bent rails, and sand boils were also evident. However, the bridges that failed were designed for less than 0.03 g, and the possibility of the separation of cantilevered spans at expansion joints was a design oversight that contributed to the bridge failures. Other nearly catastrophic structural failures in the epicentral region would justify the assignment of an epicentral MMI IX-X but not XI from ground motion intensity attenuation studies for this earthquake. However, the acceleration

FIGURE 7-16. EARTHQUAKE INTENSITY ATTENUATION CURVES FOR UPPER MISSISSIPPI BASIN NORMALIZED TO EPICENTRAL INTENSITY I_0 (See Chapter 1)

levels were extremely intense in the epicentral region of this earthquake.

The presence, or absence, of structures in different areas affected by an earthquake should not be overlooked when attempting to estimate the rate of attenuation of ground motion from an isoseismal map. For example, a nearly symmetric radiation pattern relative to the epicenter is indicated for the MMI V isoseismal for the 1971 San Fernando earthquake (see Fig. 7-6), but a very unsymmetrical pattern relative to the epicenter is indicated for the MMI VI and VII isoseismals. However, the average peak acceleration at Stations 46, 47, 48, and 52 (see Table 2, AA, 1978b) was 0.03 g for all four stations, and all stations are relatively close to the 100 km arc drawn about the epicenter. The average peak acceleration at Stations 31, 35, and 49 is 0.10 to 0.11 g, and all stations are relatively close to the 50 km arc drawn about the epicenter. This suggests that the energy radiation pattern was much more symmetrical than indicated by the isoseismal lines that depict damage. Part of the explanation lies in the fact that the area to the north of the epicenter of this earthquake is mountainous, the population density is much lower than to the south, and the number of structures available to suffer damage also was much less. This condition no doubt contributed to the dissymmetry of the resulting isoseismal lines. Since many acceleration records were available for this earthquake, it does not appear unreasonable to conclude that for ground motion attenuation studies, isoseismals VII, VI, and V can be taken as distances of 50, 100, and 250 km, respectively.

A final factor to consider when comparing intensity data for different earthquakes is the duration of the strong motion, which is a function of earthquake magnitude. For example, 30 sec of ground motion with a peak acceleration of 0.2 g may easily do the same

damage to one- and two-story masonry structures as 5 to 10 sec of ground motion with a peak acceleration of 0.3 g. No published correlation between MMI and peak ground acceleration has considered this variable, although its importance was noted by Gutenberg and Richter (1956). Thus, a lower MMI/peak acceleration attenuation relationship should be expected for a shorter duration earthquake than for a longer duration earthquake of equal magnitude.

7.3.3 CORRELATION OF MODIFIED MERCALLI INTENSITY WITH PEAK GROUND ACCELERATION

There are more than a dozen published relationships between MMI and peak horizontal ground acceleration. Unfortunately, more than an order of magnitude difference in peak ground acceleration is indicated by the different relationships. Much of the deviation has resulted because investigators have not adjusted the published MMI data so that it better represents the rate of attenuation of peak ground acceleration. Also, data have been lumped together solely on the basis of MMI, which permits duration of strong shaking and distance to be uncontrolled variables. There are two additional factors that indicate that MMI/peak horizontal ground acceleration data are not being correctly used. One is the practice of most investigators of incorrectly linearizing the nonlinear semilog relationship that exists between MMI and peak horizontal ground acceleration. The second factor is the MMI distance reference base. The same distance reference base was not used by some investigators for the MMI/peak horizontal ground acceleration relationship as is used by engineers in practice to develop the MMI/distance attenuation relationship. A brief review of relationships developed by Gutenberg and Richter (1942, 1956), Neumann (1954), Trifunac and Brady (1975),

and Murphy and O'Brien (1977) demonstrates these problems.

7.3.3.1 Gutenberg and Richter

The first Gutenberg and Richter relationship was published in 1942 when few acceleration records were available. The correlation was actually based on only 10 records for stations having Mercalli Intensities of V or greater. One point was available for Intensity VII and two for an estimated VII + 1/2. One of the latter was for the May 18, 1940 El Centro earthquake but only about one-half of the peak recorded acceleration was used, for reasons that are not explained. It is important to note that data were plotted at half-intensity intervals. Fifteen additional data points were available for Intensities I through IV + 1/2. These data points were for stations located great distances from the epicenters, and represent intensities of little significance to engineering studies. Apparently, to provide a correlation with magnitude, peak accelerations were computed for the epicenters of the earthquakes to provide 13 additional points. Based on these few records and the computed data, the semilog straight line relationship given in Figure 7-17 was derived and remained in wide use until 1975. The equation is:

$$\log_{10} A = \frac{I}{3} - 0.5 \quad (7-9)$$

where A is the maximum ground acceleration in cm/sec² and I is the Modified Mercalli Intensity.

Because Gutenberg and Richter plotted data at half-intensity intervals, full intensity values plotted on isoseismal maps would logically represent stations on or very near to isoseismal lines and half-intensity values would represent stations near the midpoint between two isoseismal lines. This is an important point, since later investigators (Trifunac and Brady, 1975; Murphy and O'Brien, 1977) have considered only full intensity intervals and have considered

any station within a given intensity area (i.e., the area lying between two isoseismal lines) to have the full intensity of the area. Actually, what these investigators have considered to be an MMI would be considered to be an MMI + 1/2 by the Gutenberg and Richter practice. This is a very important difference, since MMI distance attenuation relationships are also based on average distances from the assumed epicenter to the isoseismal lines, similar to the practice of Gutenberg and Richter. Failure to recognize this difference in reference base results in predicted accelerations that are about 40% too high when using Trifunac and Brady or Murphy and O'Brien, MMI/peak ground acceleration relationships in conjunction with MMI/distance attenuation curves that assign the MMI to the isoseismal line.

Apparently few have read a second paper by Gutenberg and Richter (1956) which redefined the 1942 relationship between Mercalli Intensity and peak ground accelerations and was based on much more data. Data were compiled on 61 additional earthquakes for which as many as 8 to 14 separate stations recorded information for the larger events. Earthquake magnitudes varied from 4.1 to 7.6, but only five accelerograph stations experienced an Intensity VII, with most of the records in the IV to V range. Epicentral accelerations were again computed, rather than measured, to obtain values for Intensities of VII to X. The conclusion in the 1956 paper was that the previously derived equation (which was so widely used until 1975) "fails for the higher intensities." In lieu of an equation, a table of values was given, since the relationship was found to be decidedly nonlinear on a semilog plot. This relationship is plotted in Figure 7-17.

The engineering profession has apparently accepted a linearization of a nonlinear relationship, for convenience. Equation 7-9 actually

FIGURE 7-17. CORRELATIONS BETWEEN PEAK HORIZONTAL GROUND ACCELERATION AND MODIFIED MERCALLI INTENSITY

provides an acceleration at Intensity X that is 50% greater than given by the Gutenberg-Richter 1956 relationship, but at Intensity VIII gives a peak ground acceleration that is 50% less. Since the peak ground acceleration/distance relationship is nonlinear on either a semilog or log-log plot at distances less than 20 km from the epicenter (see Fig. 7-8; MMI can be taken as analogous to peak ground acceleration) the Gutenberg-Richter 1956 nonlinear relationship is more logical than the earlier linear relationship (1942). There are still two uncontrolled variables that affect the 1956 relationship that should not be overlooked: the duration of strong ground motion, which can be related to magnitude, and the distance from the epicenter.

7.3.3.2 Neumann

Neumann (1954) related MMI and epicentral distance with the period and peak ground acceleration of recorded ground motions prior to the Gutenberg-Richter 1956 study. The Neumann study assumed a point source for the energy release and assumed that the shortest distance from the epicenter to a given intensity (i.e., most rapid rate of attenuation) indicated attenuation distances for granitic rock. Both assumptions are difficult to justify, especially for large magnitude earthquakes. However, the study also associated the peak ground acceleration with the predominant period of the recorded motions and recognized that the period of the peak ground acceleration changes with distance from the epicenter. These were important contributions; however, the method was never extended as more field measurements became available, and the relationships by Neumann have never had wide use.

7.3.3.3 Trifunac and Brady

Following the San Fernando 1971 earthquake, Trifunac and Brady (1975) published a

correlation of MMI with peak horizontal ground acceleration. Slightly more than half of the records used (98 stations, 195 horizontal components) were associated with the 1971 San Fernando earthquake. The resulting equation, plotted in Figure 7-17, is:

$$\text{Log}_{10} A = 0.014 + 0.30 I \quad (7-10)$$

with the same nomenclature as for Equation 7-9. Since it was an up-to-date correlation, it attracted wide use and quickly replaced the Gutenberg-Richter 1942 relationship. While the Trifunac-Brady relationship has scientific value, there are several factors which restrict its usefulness for engineering studies, but of equal importance, it is generally not correctly used by the engineering profession.

Factors affecting the usefulness of the Trifunac-Brady relationship are (1) it is based on deficient data; (2) the data available indicate a strong nonlinear relationship on a semilog plot, similar to the data used by Gutenberg and Richter (1956), but a linear extrapolation of the data was used; (3) all data for the same MMI were lumped together with no consideration of magnitude or epicentral distance; and (4) since the objective was to use MMI as an indicator of strength of motion (i.e., peak ground acceleration), obvious adjustments in MMI were required but not made.

The Trifunac-Brady MMI/peak horizontal ground acceleration relationship is indicated to be applicable to MMI IV through X, but there was adequate data for only three intensity levels, MMI V, VI, and VII. At these intensities, there were 66, 134, and 150 peak acceleration data points (i.e., 33, 67, and 75 stations), respectively, but at MMI IV there were only 6 data points (3 stations) and at MMI VIII there were only 12 data points (6 stations). At MMI IX there was no data, and at MMI X there was only the Pacoima Dam record. Hence, a deficiency of data.

The nonlinear relationship of the Trifunac and Brady data is indicated in Figure 7-17, which shows the average accelerations for MMI IV through VIII. A solid line connects MMI V through VII for which there is ample data, but a broken line connects both MMI IV and VIII to indicate that peak accelerations for these two intensities are based on inadequate data. The strong nonlinear relationship indicated by the intensities for which there is ample data confirms the Gutenberg and Richter conclusion in 1956 that the relationship is nonlinear on a semilog plot. Since data for one station (MMI X, Pacoima Dam) should not outweigh the nonlinear trend of data from 175 stations (MMI V, VI, and VII), there is little rational basis for Equation 7-10.

No investigator to date has attempted to consider the effects of duration of shaking on the MMI, and only Neumann and Murphy and O'Brien have attempted to consider distance as a variable. This has no doubt resulted from deficient data. However, it would appear more logical to construct MMI/peak horizontal ground acceleration relationships for each earthquake for which there is data, than to lump data for the same MMI from all earthquakes together. There is also a strong need to adjust MMI values to better reflect the true intensity of ground motion in terms of peak ground acceleration. These two objectives can be accomplished by making a detailed study of each earthquake from which acceleration data is used. The study of each earthquake should include independent correlations of peak horizontal ground acceleration and MMI with distance, and the development of an MMI/peak horizontal ground acceleration relationship which is based on adjusted MMI values that better reflect attenuation in strength of ground motion, than MMI values that reflect observed damage. Only in this way can curves be

developed for different magnitudes. The procedure recommended can be demonstrated for the 1971 San Fernando earthquake.

A correlation of peak ground acceleration with distance for the 1971 San Fernando earthquake was discussed in Section 7.2.2.1. (See also Tables 2 and 3 in AA (1978b) for recorded accelerations and other information for this earthquake.) The next step is to adjust MMI values and then establish an adjusted MMI/distance relationship and finally an MMI/peak ground acceleration relationship for this earthquake. However, there are some shortcomings to merely tabulating assigned intensities and computing an average peak horizontal ground acceleration, which is the normal practice.

In Figure 7-5, the 55 recording stations classified as MMI VII for the 1971 San Fernando earthquake are represented by Stations 2 through 23 plus Station 28. Note that Station 24 falls inside the MMI VII isoseismal and was ranked MMI VI, but Station 28 falls outside the MMI VII isoseismal and is ranked MMI VII. This is not unusual near an isoseismal boundary. Also, note that Stations 9, 12, 15, 17, 18, and 19 collectively represent 36 stations (72 components) that are all located about 40 km from the epicenter. Stations 13, 14, 20, 21, 22, and 28 (12 components) are also about 40 km from the epicenter. Thus 42 stations, or 84 out of 110 data points, are all located within 41 ± 2.3 km from the epicenter. The procedure in the past has been to compute an average for the MMI VII area using these data points. The data points are obviously too poorly distributed for this earthquake to be used to compute an average for the area.

In Figure 7-6, Stations 25, 26, 27, 28, 29, and 30 have been assigned MMI VI, but acceleration records indicate that these stations experienced accelerations as strong as stations at the same distance to the south of the epicenter that

were ranked a strong VII. The probable reason is that these stations are in a less densely populated area and there were few structures in this area that could be damaged. The importance of these stations in computing the average peak horizontal ground acceleration is demonstrated in Figure 7-7 where the peak horizontal ground accelerations have been indicated by symbols that are keyed to the assigned MMI values. Note the great number of MMI VI data points plotted between the vertical MMI VII-VIII isoseismal distances. The same condition exists for Stations 49, 50, 51, and 52, which have been assigned MMI V, but the peak accelerations for these stations and the epicentral distances are comparable to MMI VI. Note the MMI V data points plotted between the MMI VI and VII vertical isoseismal distances in Figure 7-7.

These adjustments could be made and a more representative average computed for each isoseismal area, but a simpler solution is to locate the average distances to each isoseismal line on Figure 7-7 and then read off the peak horizontal ground acceleration from the correlation curve of acceleration versus distance. For this procedure, peak horizontal ground accelerations of 0.083, 0.033, and 0.012 g, respectively, are obtained for MMI VII, VI, and V. Selecting a peak horizontal ground acceleration for MMI VIII is more difficult as there are no acceleration data for epicentral distances less than 20 km except for the Pacoima Dam record. Because of the absence of data, it would be better to not define accelerations for MMI values greater than VII for this earthquake. These values for MMI V, VI, and VII have been plotted in Figure 7-17 and contrast strongly with the averages indicated by Trifunac and Brady. Part of this difference results from the fact that the Trifunac and Brady data represent the average acceleration for the data points in the areas between isoseismal lines, but the

values taken from Figure 7-7 represent the average acceleration at the distance that corresponds to the isoseismal line. The close agreement of the values taken from Figure 7-7 with the Gutenberg and Richter (1956) curve in Figure 7-17 should also be noted.

7.3.3.4 Murphy and O'Brien

Murphy and O'Brien (1977) have provided an interesting correlation between MMI and peak ground acceleration that includes acceleration/MMI data from 900 western U.S. (Trifunac and Brady data), 500 Japanese, and 60 southern Europe accelerograph stations. In comparison to the Trifunac and Brady study, adequate data were available for MMI III through VII, with 18 data points for MMI VIII, none for IX, and 2 (Pacoima Dam) for X. This study indicates that a different MMI/peak ground acceleration relationship exists for Japanese, U.S., and European earthquakes. It also demonstrates a relationship for the 1971 San Fernando earthquake between peak horizontal ground acceleration, MMI, and epicentral distance. Also, for a fixed distance of 30 km, it demonstrates that MMI/peak ground acceleration can be expressed as a function of magnitude. However, neither of these relationships has been developed to the point that they can be used for prediction purposes.

Using the entire data sample, and with magnitude and distance uncontrolled variables, Murphy and O'Brien indicate a linear relationship (on a semilog plot) between peak horizontal ground acceleration and MMI that is in agreement with the Trifunac and Brady linear relationship (on a semilog plot) at MMI X, but the relationship is slightly lower than the Trifunac and Brady relationship at all other intensities. Three important factors relative to the major correlations by Murphy and O'Brien are: (1) data have again been lumped by MMI

irrespective of magnitude and distance, (2) the major correlations are based on world earthquakes, which means from regions that have different attenuation characteristics, and (3) although the data clearly indicate a non-linear relationship between MMI and peak ground acceleration, the authors chose to fit a linear relationship (on a semilog plot) to the data. It may not be possible to correct the first factor, due to lack of data to develop peak horizontal ground acceleration/distance relationships for individual earthquakes; and to modify such relationships from a world-average to a specific site requires considerable effort. However, the third factor – the nonlinearity (on a semilog plot) of the data – can be easily corrected.

The mean accelerations determined by Murphy and O'Brien for all data have been plotted in Figure 7-18 to demonstrate the non-linearity (on a semilog plot) of the data. Since the average peak horizontal ground accelerations are again averages for the areas between isoseismal lines, Murphy's data have been offset one-half an intensity level to the right so that it can be compared with the Gutenberg and Richter (1956) relationship and the data points determined in Figure 7-7 for the 1971 San Fernando earthquake. The close agreement at MMI VI through VIII with the Gutenberg and Richter (1956) curve and the San Fernando data is important. Higher accelerations at intensities below VI cannot be explained without careful study of all the data. One possible explanation is that this is contributed by the Japanese data, and signifies the effect of greater focal depths than typical of California data. Fortunately, this is not of great significance for nuclear power plant design since the minimum peak horizontal ground acceleration considered in design is 0.10 g, which corresponds to MMI VII. It should be noted that Murphy and O'Brien use only 18 data points for MMI VIII,

which does not provide a reliable value for this intensity.

7.3.3.5 Conclusions

Both the Trifunac and Brady study and the Murphy and O'Brien study used accelerations associated with the areas between isoseismal lines. In contrast, Gutenberg and Richter utilized half-intensity values, which means that their MMI intensity base of reference is associated with the isoseismal lines rather than with the area between isoseismal lines. When this adjustment is made to the Murphy and O'Brien data, close agreement is obtained with the Gutenberg and Richter (1956) relationship except for MMI V and below. However, these lower intensity levels are not of significance to nuclear power plant design.

Adjustment of the 1971 San Fernando data used by Trifunac and Brady to better reflect the rate of attenuation of acceleration, and to represent accelerations at the boundaries (i.e., at the isoseismal lines) of the individual intensity areas also gives values that are in close agreement with the Gutenberg and Richter (1956) relationship. Therefore, for intensities of MMI VIII and below, the Gutenberg and Richter (1956) relationship gives the best average values for California earthquakes. There are not sufficient data to verify the Gutenberg and Richter relationship at MMI IX and above. Additional studies are required to determine whether earthquake magnitude (i.e., duration) and distance can be introduced as variables in the MMI/peak horizontal ground acceleration correlation.

For regions where no strong motion acceleration records have been recorded, it is recommended that MMI/distance attenuation relationships be developed based on regional historical earthquakes and compared with MMI/distance attenuation relationships for historical

FIGURE 7-18. MMI/PEAK HORIZONTAL GROUND ACCELERATION RELATIONSHIPS

California earthquakes. If the attenuation relationships are comparable, then the Gutenberg and Richter (1956) MMI/peak horizontal ground acceleration correlation should be applicable for MMI values of VIII or less. If significantly different rates of attenuation exist between the MMI/distance relationships based on historical earthquakes, then adjustments in the Gutenberg and Richter (1956) MMI/peak horizontal ground acceleration relationship may be required. The adjustments must be based on special studies for each particular region, and may require judgmental decisions because of the absence of quantitative data.

In many regions of eastern United States, nuclear power plant design will require that the design be based on epicentral MMI values of VII to VIII. Attenuation relationships will therefore not be required. For regions where epicentral intensities of MMI VII to VIII apply, and there is no evidence of surface faulting, the Gutenberg and Richter (1956) MMI/peak horizontal ground acceleration relationship gives reasonable estimates for rock or firm soil site conditions. However, the Gutenberg and Richter values should be assumed to apply to the respective MMI isoseismal lines, rather than to midpoints of areas between isoseismal lines.

CHAPTER 8

PEAK GROUND ACCELERATION IN THE VERY NEAR FIELD

8.1 INTRODUCTION

There are numerous correlations depicting the attenuation of earthquake vibratory ground motion as a function of peak ground acceleration, earthquake magnitude, and source distance (Donovan, 1973; Schnabel and Seed, 1973; Trifunac, 1976; McGuire, 1978; and Werner, Ts'ao, and Rothman, 1979). These correlations have been based on data recorded at source distances that in general are greater than 20 km. As a result, there is reasonable agreement between correlations at source distances greater than 20 km but poor agreement in the very near field (i.e., source distances less than 20 km). Since nuclear plants are frequently located closer than 20 km to active faults, it is important that ground motion prediction procedures be improved in the very near field.

A pilot study was conducted in which a regression analysis was based on ground motion data recorded at hypocentral distances that varied from 7 to 39 km, with most of the data being recorded at hypocentral distances less than 20 km. This analysis differs from the previous efforts in that the peak ground acceleration was expressed as a function of tectonic stress drop and fault dimension as well as a function of the conventional magnitude and distance parameters. The regression analysis is based on 58 earthquake events of which 12 were recorded during and following the 1940 El Centro, California, earthquake; 13 were recorded following the 1971 San Fernando, California, earthquake; and 33 were recorded following the May 6, 1976 Friuli, Italy, earthquake.

8.2 DATA BASE AND ANALYSIS PROCEDURE

8.2.1 SOURCE PARAMETERS

Most conventional correlations have expressed peak horizontal ground acceleration as a function of only magnitude and distance. Now it may be appropriate to also include the tectonic stress drop and a fault dimension defining the most highly stressed zone of rupture (Bernreuter, 1977). These values are computed from Fourier amplitude spectra using the dislocation model developed by Brune (1970). The spectra are computed from strong motion acceleration records monitored in the very near field. The magnitude of an earthquake is also determined from seismograph records but is based on independent measurements recorded by Wood-Anderson torsion seismometers at source distances greater than the source distances of the strong motion recording stations. A detailed derivation of the procedure to compute the stress drop and fault dimension can be found in Brune (1970) and Trifunac (1972a).

In Brune's dislocation model, the average free-field spectrum for the shear wave motion is expressed as

$$\Omega_s(\omega) = \frac{r}{R} \frac{\Delta\sigma\beta}{\mu} \frac{R_{\theta\phi}^S}{\omega^2 + \omega_C^2} \quad (8-1)$$

where

$\Omega_s(\omega)$ = Fourier amplitude spectra for shear wave motion

ω_C = Corner frequency

$$\left(\frac{7\pi}{4}\right)^{1/2} \cdot \frac{\beta}{r} \quad (8-2)$$

R = Distance from source to station

- r = Radius of the circular dislocation (fault dimension)
 $R_{\theta\phi}^S$ = Radiation pattern of the S-wave
 β = Shear wave velocity
 μ = Rigidity of the rocks where earthquake occurs (Lamé constant)
 $\Delta\sigma$ = Tectonic stress drop

When $\omega \rightarrow \bar{0}$, $\Omega_s(0)$, the amplitude of the long-period asymptote of the S-wave spectrum, can be related to the seismic moment M_0 (Keilis-Borok, 1960):

$$\Omega_s(0) = R_{\theta\phi}^S M_0 (4\pi\mu R\beta)^{-1} \quad (8-3)$$

where M_0 is defined (Maruyama, 1963) as

$$M_0 = \mu \bar{u} \pi r^2 \quad (8-4)$$

and $\bar{u}$ is the average dislocation.

With this theoretical relationship among the source parameters $\Delta\sigma$, r , M_0 , and $\bar{u}$, the following steps are used to compute these quantities:

- With the Cooley-Tukey Fast Fourier algorithm, the Fourier amplitude spectra are calculated from very near field acceleration records. Spectra for the two horizontal components are vectorially combined to arrive at the average S-wave acceleration spectrum.
- The theoretical spectra (Eq. 8-1) is multiplied by ω^2 and fitted to the calculated spectra.
- The corner frequency ω_c obtained from Step b is used to compute fault dimension r (Eq. 8-2)

$$r = \left(\frac{7\pi}{4} \right)^{1/2} \frac{\beta}{\omega_c}$$

- Combining Equations 8-1, 8-2, 8-3, and 8-4, the remaining parameters are

$$M_0 = \frac{\Omega_s(0) 4\pi\mu R\beta}{R_{\theta\phi}^S}$$

$$\Delta\sigma = \frac{7}{16} \frac{M_0}{r^3}$$

$$\bar{u} = \frac{M_0}{\mu \pi r^2}$$

In performing the statistical analysis for this study, only $\Delta\sigma$ and r were used because the other parameters M_0 and $\bar{u}$ were related to $\Delta\sigma$ and r . Similar expressions can be derived for P-wave spectra; however, P-wave spectra were not used here (see Sec. 8.2.2).

Magnitude, stress drop, and fault dimension are related, and a correlation for these three parameters has been provided by Thatcher and Hanks (1973) based on California earthquakes. Thus, magnitude can be eliminated from the regression analysis by the adoption of this correlation. However, it was decided that this would not be done here for the following reasons:

- The data for magnitude is an independent determination based on Wood-Anderson torsion seismometer measurements, while the stress drop and fault dimension are derived terms based on strong motion instrument measurements. Therefore, the independence of the two sets of data could be better maintained by including all three parameters in the regression analysis.
- The correlation of magnitude, stress drop, and fault dimension is empirical. A change in the data base, as in this study, results in a different correlation than found by Thatcher and Hanks. A correlation based on the data used in this study resulted in a standard deviation of 0.624. However, the standard deviation for the final regression equation for peak ground acceleration that included the three variables was less (0.485). Thus, although a correlation function was developed, the reliability of the

function is low and the regression analysis would not have been improved by using a correlation function of low reliability to eliminate one of the parameters.

- c. A step-down algorithm was used to conduct the significance test of each parameter. The results (Table 8-1) indicated that magnitude, stress drop, and fault dimension were all significant and should be included in the regression equation.
- d. If magnitude is included, a comparison can be provided between the regression equation developed here and conventional correlation equations that express peak ground acceleration as a function of only magnitude and distance.

8.2.2 DATA SOURCES

Processed acceleration records from the three source earthquakes are listed in Table 8-1. The data from the 1940 El Centro earthquakes were compiled by Trifunac (1972a) and are important because this set includes the major earthquake events and the aftershocks. The 1940 El Centro earthquake consisted of four separate events during the main release of energy, followed by nine aftershocks. The four main events represented earthquake magnitudes of 6.0, 6.1, 5.8, and 6.5 and occurred within a 25-sec period. Of the nine aftershocks, one was not used because the amplitude of the record was too small to read. Hence, 12 events are listed in the table. Hypocentral distances varied from 12 to 39 km.

Trifunac (1972b) has also compiled data for 35 aftershocks recorded at the Pacoima Dam following the main energy release of the February 9, 1971 San Fernando earthquake. Thirteen of these records were used by

Trifunac for further study and are listed in the table. The hypocentral distances varied from 7 to 18 km for these events.

The Friuli region in Italy experienced a major earthquake on May 6, 1976. Between that date and October 1977, a total of 250 recordings of 54 shocks were obtained (described by Basili et al., 1977a). These records from four stations (Majano, Forgario-Cornino, S. Rocco, and Buia) have been used to estimate the source parameters (Basili et al., 1977b). A total of 33 sets of data were compiled, as shown in the table. Source distances varied from 7 to 26 km, and earthquake magnitudes as great as 6.1 were experienced.

Although source parameters were computed based on both P-wave and S-wave acceleration spectra for the California earthquakes, only the data computed from the S-wave were used in this study since only S-wave data were available from the Friuli earthquakes.

Correct procedures used to process the strong motion records are important. The California records were processed by procedures described by Hudson (1974), which requires a digitization of the field accelerograph records using semi-automatic or fully automatic image scanning processes. The numerical corrections – an instrument correction and a baseline correction – are then applied to the digitized data. The instrument correction first consists of a low-pass filtering of the original data to eliminate high-frequency digitization errors and aliasing; then, a differential equation of motion for a damped single-degree-of-freedom system with a high natural frequency is applied to the data to recover higher frequency components of motion that have been filtered by the transducer (Trifunac, 1972c). The baseline correction consists of the use of a high-pass filter that filters out periods longer than 16 sec and eliminates the long period errors caused by

TABLE 8-1. LIST OF RECORDS USED IN THIS STUDY

No.	Events	Hypocentral Distance, km, R	Fault Dimension, km r	Stress Drop, Bars $\Delta\sigma$	Magnitude, M	Peak Acceleration, 0.1 g		
						Horiz. a_H	Vert. a_V	Horiz. a_H
1	El Centro	14.0	2.30	177.0	6.0	3.67	2.71	2.07
2		15.0	1.50	188.0	6.1	1.91	1.19	1.45
3		16.0	1.80	143.0	5.8	1.49	0.69	1.23
4		35.0	3.20	349.0	6.5	1.53	0.57	1.53
5		24.0	1.20	63.0	5.0	0.268	0.09	0.256
6		18.0	1.30	16.0	4.4	0.09	0.114	0.153
7		12.0	1.10	12.0	4.4	0.128	0.102	0.243
8		39.0	1.20	103.0	5.4	0.243	0.077	0.114
9		30.0	2.20	23.0	4.8	0.077	0.159	0.128
10		16.0	1.70	57.0	5.2	0.740	0.511	0.817
11		17.0	0.8	30.0	4.7	0.165	0.128	0.255
12		33.0	1.20	217.0	5.9	0.55	0.128	0.842
13	San Fernando	18.0	0.88	502.0	5.5	0.90	0.80	1.20
14		12.0	0.56	30.0	4.3	0.20	0.10	0.20
15		15.0	0.36	227.0	4.9	0.50	0.30	0.60
16		13.0	0.42	72.0	4.6	0.30	0.20	0.40
17		11.0	0.56	29.0	4.4	0.20	0.10	0.20
18		9.0	0.22	111.0	4.4	0.40	0.10	0.40
19		12.0	0.74	98.0	4.8	0.90	0.10	0.90
20		12.0	0.45	307.0	5.4	1.20	0.30	1.30
21		7.0	0.73	29.0	4.4	0.60	0.20	0.40
22		11.0	0.56	23.0	4.3	0.10	0.10	0.10
23		12.0	0.63	69.0	4.6	0.30	0.30	0.30
24		17.0	0.69	48.0	4.6	0.20	0.10	0.30
25		14.0	0.93	20.0	4.3	0.10	0.10	0.20
26	Friuli	25.0	5.00	34.0	5.5	0.84	0.29	0.48
27		25.0	4.80	47.0	5.5	0.43	0.26	0.37
28		20.0	5.00	17.0	4.7	0.48	0.14	0.38
29		21.0	6.00	11.0	4.7	0.31	0.14	0.29
30		22.0	8.00	18.0	5.3	1.90	1.68	3.08
31		7.0	5.00	26.0	4.3	0.74	0.61	0.66
32		26.0	3.40	39.0	4.2	0.15	0.10	0.28
33		12.0	4.80	27.0	4.3	0.59	0.42	0.62
34		7.0	2.40	63.0	3.7	0.23	0.22	0.26
35		10.0	3.80	11.0	3.4	0.14	0.09	0.08
36		9.0	4.00	37.0	4.2	0.27	0.16	0.25
37		14.0	6.00	15.0	4.5	0.47	0.28	0.44
38		14.0	7.00	15.0	4.2	0.72	0.45	0.64
39		12.0	4.60	12.0	3.3	0.11	0.08	0.13
40		16.0	5.00	17.0	4.4	1.00	0.56	0.89
41		16.0	5.00	29.0	4.4	0.69	0.31	0.35
42		13.0	5.40	50.0	4.4	0.49	0.29	0.53
43		11.0	6.00	35.0	4.4	0.33	0.19	0.24
44		18.0	6.50	30.0	5.5	0.95	0.51	1.15
45		18.0	6.30	29.0	5.5	0.42	0.20	0.09
46		17.0	6.50	81.0	5.9	1.33	1.19	2.35
47		17.0	6.00	44.0	5.9	0.92	0.48	0.91
48		14.0	6.50	42.0	5.9	2.34	0.93	1.14
49		24.0	5.40	28.0	4.4	0.26	0.20	0.16
50		16.0	8.50	38.0	6.1	2.63	0.99	2.18
51		15.0	8.50	42.0	6.1	0.69	0.59	1.19
52		12.0	8.50	33.0	6.1	1.10	0.82	1.02
53		24.0	7.10	16.0	5.0	0.58	0.35	0.54
54		24.0	6.50	23.0	5.0	0.39	0.16	0.33
55		23.0	6.00	97.0	6.0	3.53	1.95	3.36
56		23.0	5.20	115.0	6.0	1.45	0.83	2.28
57		19.0	5.20	115.0	6.0	0.85	0.87	1.01
58		20.0	6.00	16.0	4.1	0.29	0.22	0.44

recording, digitization, and record processing (Trifunac, 1971). The final result is a corrected acceleration time history. Other ground motion data, such as displacement, velocity, response spectra, Fourier amplitude, and peak ground motion values, are all derived from the

processed time-history record. Processing procedures for the Italian earthquake records are comparable to these methods (see Basili et al., 1977a). The regression analysis procedure is derived in AA (1980b) and will be used in the following section.

8.3 RESULTS AND RECOMMENDATIONS

A regression equation is used here, and results are compared with conventional relationships that include only magnitude and source distance as parameters. This regression analysis procedure is then applied to the California and Friuli earthquakes in order to achieve research recommendations.

8.3.1 REGRESSION EQUATION

The regression analysis procedure used here (see AA, 1980b) resulted in Equation 8-5 and constants for peak ground acceleration; both are listed in Table 8-2. There are several uncertainties in the analysis, as follows:

- a. The near-field data were limited to 58 sets (i.e., 116 horizontal components and 58 vertical components).
- b. The stress drop and the fault dimension were computed based on an approximate dislocation model. The fault dimension is the equivalent radius of the surface of the most highly stressed region. This appears to be a very uncertain dimension (see Figs. 8-1b and 8-4b).
- c. A combination of Italian and California earthquake data was used in the regression analysis. The effect of this nonuniformity of data base is undetermined.
- d. Because of the lack of a complete theoretical relationship, the regression analysis has been restricted to a simple linear form (that is, the parameters or their transformations are linearly related).

In the regression equation itself, the distance term in $\ln(R + \Delta)$, where $\Delta = 1$, was adopted from the study by Werner, Ts'ao, and Rothman (1979). The basis for using $\Delta = 1$ was

TABLE 8-2. REGRESSION EQUATION FOR PEAK GROUND ACCELERATION

$\ln A = a_1 + \frac{a_2}{M} + a_3 \ln(R + 1) + a_4 E$ $+ a_5 \ln(R + 1) E + a_6 \ln(\Delta\sigma)$ $+ a_7 \ln(r) + a_8 \ln(R + 1) V$ <div style="text-align: right;">(8-5)</div>		
where		
A	=	Peak ground acceleration in 0.1 g
M	=	Local magnitude
R	=	Hypocentral distance, km
V	=	$\begin{cases} 1 & \text{(for vertical motion)} \\ 0 & \text{(for horizontal motion)} \end{cases}$
E	=	$\begin{cases} 1 & \text{(for Friuli earthquakes)} \\ 0 & \text{(for California earthquakes)} \\ 0.57 & \text{(for combined Friuli and California earthquakes)} \end{cases}$
$\Delta\sigma$	=	Tectonic stress drop in bars
r	=	Radius of equivalent circular dislocation (fault dimension), km
$a_1 \dots a_8$	=	Regression coefficients
Regression Coefficients	Value	Significance
a_1	3.0512	0.9999
a_2	-9.2358	1.0
a_3	-1.5963	1.0
a_4	-4.3348	1.0
a_5	1.2851	1.0
a_6	0.5848	1.0
a_7	0.9071	1.0
a_8	-0.1946	1.0

determined by conducting a parametric study of values Δ . It was found that when $\Delta = 1$, the regression equation best fits the source data compiled by Trifunac (1972a, b) and Thatcher and Hanks (1973). However, the hypocentral distance was used here, rather than the epicentral distance used in the Werner, Ts'ao, and Rothman study. Second, the justification for using the magnitude term as a_2/M instead of $a_2 M$ is as follows: When M approaches 0, the right-hand side of Equation 8-5 approaches $-\infty$ (a_2 is always negative), so the acceleration A reaches zero. On the other extreme, when the magnitude approaches infinity, the right-hand side of Equation 8-5 approaches a constant,

which in turn gives rise to a constant acceleration. This is supported by the belief that as the magnitude increases, the acceleration becomes less sensitive to variations in magnitude.

Data points that deviated strongly from the regression equation were identified in the following manner. The difference between the log of the peak ground acceleration and the computed regression curve was compared with the standard deviation of the data (expressed as a logarithm). When the difference was found to be greater than 2.5 times the standard deviation, the data point was identified as an outlier. All the data used in the regression analysis were checked, and the outliers listed in Table 8-3. The four records listed in Table 8-3 were checked further, but no errors were detected.

8.3.2 COMPARISON WITH CONVENTIONAL CORRELATION AND RECENT EPICENTRAL ACCELERATION DATA

The inclusion of magnitude in the regression relationship (Eq. 8-5) permits a comparison between this equation and earlier conventional correlations that consider the peak ground acceleration to be a function of only magnitude and distance. In order to do this, reasonable values must be defined for the stress drop and the fault dimension in Equation 8-5. Before this comparison could be made, additional study was required of the relationship between magnitude, stress drop, and fault dimension. The

comparison using the El Centro data (see Table 8-1) will be expanded to include epicentral peak ground accelerations recorded during the October 15, 1979 El Centro earthquake.

The El Centro data for stress drop and fault dimension (Table 8-1) are plotted in Figure 8-1 as functions of earthquake magnitude. The plot for stress drop indicates that a reasonably stable relationship exists, except that the shape of the curve cannot be defined above magnitude 6.5; it is possible that this curve becomes horizontal at magnitudes of 7.5 to 8.5. The 1979 El Centro earthquake has an assigned magnitude of 6.5, which is within the range of the data. A stress drop of 350 bars was selected as a reasonable value for a magnitude 6.5 earthquake in the Imperial Valley region.

The plot for fault dimension is much less definitive than the relationship between magnitude and stress drop (see Fig. 8-1b). Below magnitude 6 the scatter in data is quite large. Above magnitude 6 it is believed to be more stable; however, there is little data at this magnitude level. A value of 3.2 km, which was computed for the 6.5 magnitude 1940 El Centro earthquake, was selected as a reasonable value. This value is in close agreement with Figure 8-1b.

Figure 8-2 provides the horizontal peak ground accelerations resulting from Equation 8-5 when evaluated for an earthquake magnitude of

TABLE 8-3. OUTLIERS FROM REGRESSION ANALYSIS

Accelerograph Station		Earthquake Event			Direction: Horizontal, Vertical,	Value of Parameter, 1/10g	Deviation of $\ln A$
Location	Hypocentral Distance, km	Location	Date	Magnitude			
S. Rocco	18	Friuli	Nov 9, 1976	5.5	H	0.09	-4.5 σ
Forgaria	22	Friuli	Nov 15, 1976	5.3	H	3.08	2.7 σ
Forgaria	22	Friuli	Nov 15, 1976	5.3	V	1.68	2.6 σ
Pacoima Dam	12	San Fernando	Feb 9, 1976	4.8	V	0.10	-2.6 σ

(a) Relationship between magnitude and stress drop

(b) Relationship between magnitude and fault dimension

FIGURE 8-1. RELATIONSHIPS BETWEEN EARTHQUAKE MAGNITUDE AND STRESS DROP AND FAULT DIMENSION BASED ON 1940 EL CENTRO EARTHQUAKE DATA

6.5, a stress drop of 350 bars, and a fault dimension of 3.2 km ($E = 0$). A study by Werner, Ts'ao, and Rothman (1979) produced the following relationship for peak horizontal ground acceleration based on the San Fernando earthquake data:

$$\ln a' = 14.10 - \frac{32.85}{M} - 1.39 \ln (D + 1) \quad (8-6)$$

where

$\ln a'$ = Natural logarithm of the peak horizontal ground acceleration, in./sec²

M = Local magnitude

D = Epicentral distance, km

This equation has been recommended as a guideline with the adjustment that the hypocentral distance R be substituted for $(D + 1)$ (see AA, 1980b). This adjusted equation is plotted in Figure 8-2 for a magnitude 6.5 earthquake. Comparable values for peak horizontal ground acceleration are found for both equations. Selected peak horizontal ground accelerations recorded during the 1979 El Centro earthquake have also been plotted in Figure 8-2.

The locations of recording stations, the fault ruptures, and the epicenter are shown in Figure 8-3. This earthquake had a preliminary assigned magnitude of 6.4. It is a significant earthquake in that many records were monitored in close proximity to the fault. The recording station at Bond's Corner is particularly important in that it has an epicentral distance of only 6 km and a fault distance of 3 km. Other stations have shorter fault distances (1 km) but are more than 20 km from the epicenter. This poses the important question of whether the peak signals recorded at these stations emanated from the hypocenter or from a closer point along the fault.

On the hypothesis that the hypocenter represents the point where the most highly stressed rock first fails and releases the strongest signals, it is possible that the signal at Bond's Corner may have emanated from the epicenter. The stations at Calexico and El Centro Station 13, which have fault/epicentral distance ratios of 11/15 and 22/34, respectively, may also have received peak signals from the epicenter, although this is not certain. Peak horizontal ground accelerations for these three

FIGURE 8-2. COMPARISON OF EQUATIONS 8-5 AND 8-6 WITH OCTOBER 15, 1979 EL CENTRO PEAK GROUND ACCELERATIONS

stations have therefore been plotted in Figure 8-2 both as a function of epicentral distance and as a function of the shortest distance to the fault. Peak horizontal ground accelerations for El Centro Stations 6, 7, and 8 have been plotted only as a function of the shortest distance to the fault, as the fault/epicentral distance ratios for these stations are 1/27, 1/26, and 4/27, respectively. The peak signals in these cases would obviously have originated from the shorter fault distances.

Note in Figure 8-2 that the peak ground acceleration data from the 1979 El Centro earthquake are consistent with the postulation that the peak ground accelerations recorded at Bond's Corner resulted from the intense energy

released at the epicenter, and that at other stations the peak ground acceleration was produced by energy releases at closer points along the fault where the stress drop was lower. A lower peak horizontal ground acceleration correlation curve would therefore apply to these stations. For design, it would appear that the correlation would be more useful if based on fault distance, a longer fault dimension, and an average stress drop. Additional study is required before Equation 8-5 can be adapted to design use.

Possible ranges in stress drop and fault dimension in the data can be determined from Figure 8-1. At magnitude 6 the standard deviation indicates a range in stress drop of

FIGURE 8-3. LOCATIONS OF STRONG MOTION RECORDERS AND FAULT OF OCTOBER 15, 1979 EL CENTRO EARTHQUAKE

150 to 220 bars, while the range in fault dimension is from about 1.8 to 3.2 km. Thus, a significant range in peak ground acceleration should be anticipated for earthquakes of the same magnitude within the Imperial Valley. Likewise, one can conclude that nearly the same peak ground acceleration could be obtained from either a 5.5 or 6.5 magnitude earthquake.

8.3.3 COMPARISON OF CALIFORNIA AND FRIULI EARTHQUAKE ATTENUATION RELATIONSHIPS

The relationships between magnitude and stress drop and between magnitude and fault

dimension for the Friuli data given in Table 8-1 are shown in Figure 8-4. Much lower stress drops and larger fault dimensions were computed for the Friuli data than for the El Centro data (Fig. 8-1). The scatter in data is also much greater for the Friuli earthquakes. A comparison of the attenuation of peak ground acceleration for a magnitude 6.5 earthquake from the Friuli and El Centro data is provided in Figure 8-5. The El Centro relationship is taken from Figure 8-2. The Friuli relationship has been computed from Equation 8-5 for a stress drop of 150 bars and a fault dimension of 8.4 km (these values from Fig. 8-4). A focal

(a) Relationship between magnitude and stress drop

(b) Relationship between magnitude and fault dimension

FIGURE 8-4. RELATIONSHIPS BETWEEN EARTHQUAKE MAGNITUDE AND STRESS DROP AND FAULT DIMENSION BASED ON FRIULI, ITALY EARTHQUAKE DATA

FIGURE 8-5. COMPARISON OF PEAK HORIZONTAL GROUND ACCELERATION DISTANCE ATTENUATION RELATIONSHIPS FOR EL CENTRO AND FRIULI MAGNITUDE 6.5 EARTHQUAKES

depth of 7 km was used in the computation of the Friuli relationship given in Figure 8-5 since these earthquakes occurred at relatively shallow depths. The results indicate much lower epicentral peak ground accelerations and a lower rate of attenuation for the Friuli relationship than for the El Centro relationship.

8.3.4 RESEARCH RECOMMENDATIONS

The results of this study have led to the following recommendations for future research studies of the very near field ground motion:

a. In this study, stress drop and fault dimension were included in the regression analysis and were found to contribute significantly to the correlation of peak ground accelerations in the very near field. However, the data are limited. Future processing of data should include this type of information so that more conclusive relationships can be established. Although not investigated here, the peak ground velocity, peak ground displacement, and ground response spectrum may also be related to the source parameters, fault dimension, and stress drop, as well as magnitude.

b. The Friuli earthquake was found to exhibit characteristics quite different from that of California earthquakes. This suggests the need to analyze earthquake data on an event-by-event basis. Again, the collection of additional earthquake data is essential for such studies. California earthquakes in Coyote Lake (August 1979), El Centro (October 1979), and Santa Barbara (August 1978) have generated more near-field data that was not applied here, but should be used in future studies.

c. The peak motion data here were used without regard to the direction of motion. In future studies each set of data should be transformed to provide motion in the radial and transverse planes. In this way it may be possible to improve the quality of the data through the separation of the different waveforms present. This should be particularly true for relationships involving peak ground velocity and displacement if these motions are found to be a function of fault dimension and stress drop.

CHAPTER 9

STATISTICAL ANALYSIS OF RESPONSE SPECTRAL AMPLITUDES

9.1 ANALYSIS PROCEDURES

A regression analysis of ground response spectra is presented here in order to characterize ground motion at the near field as well as at the mid to far field. The effects of magnitude, epicentral distance, site conditions, direction of motion (whether vertical or horizontal) and specific earthquake events are included in the analysis.

9.1.1 BACKGROUND

The response spectrum plays an important role in defining seismic input criteria for the earthquake-resistant design of structures. A widely used method to derive such criteria spectra is by means of a statistical approach applied to available ground motion data. The advantage of the statistical approach is that the derivation is based on real data, and the uncertainties that are encountered in data collection, in equation assumptions, etc., can be considered by computing the scatter of data about the assumed regression equations. The ability to quantify such uncertainty is important in defining the seismic input criteria for the structure.

In the past, many statistical analyses have been performed using a wide variety of ground motion parameters. Perhaps the most widely used results from such analyses are those of Newmark, Blume, and Kapur (1973), which were adopted in the Regulatory Guide (RG) 1.60 for use in defining seismic input criteria spectra for nuclear power plants (USAEC, 1973b). However, these results have the following shortcomings:

- a. The spectra were developed on the basis of only about 50 records of

ground motion records. This quantity of data is too small to represent an adequate statistical basis for developing such spectra. Also, most of these records correspond to peak accelerations that are very low.

- b. The spectrum was specified by a set of fixed spectral amplification factors, normalized according to peak acceleration only. However, the peak acceleration is a measure of only the higher frequency components of the ground motion and is therefore not suitable for scaling low- and intermediate-frequency spectral amplitudes. Furthermore, the use of fixed spectral amplification factors cannot possibly represent the potentially important effects of magnitude, distance, site conditions, etc., on the spectrum shape (Trifunac, 1977). Therefore, the resulting spectrum shape will be the same regardless of the magnitude and epicentral distance of the design earthquake event and the subsurface geologic conditions at the site.

These shortcomings were eliminated in a statistical study (SW/AA, 1979) using multilinear regression analyses in conjunction with a much larger data base (180 sets of records with three components per set) than was used in the Newmark-Blume-Kapur study. These regression analyses incorporated the effects of magnitude, epicentral distance, and site conditions on the shape and amplitude of the ground response spectrum, and also differentiated between spectra generated during the San Fernando earthquake relative to other earthquake events.

They showed that these parameters have an important influence on the response spectrum shape and amplitude.

Another important result of the SW/AA study was that reasonable agreement was shown to exist between statistical analysis results from numerous studies involving response spectral amplitudes in the mid to far field. For near field motions (epicentral distance < 20 km), substantial scatter was shown to exist among the predictions of the various studies, which was attributed to the limited number of available near field records. In recent years, however, the number of near field records has increased substantially, and these additional data will help our understanding of the characteristics of earthquake ground motions generated near the causative fault.

9.1.2 DATA DESCRIPTION

The strong motion data used in this investigation were obtained from two main sources: California Institute of Technology (CIT) and United States Geologic Survey (USGS). Both sources used the same format in preparing the data. For each recorded ground motion, the basic information includes the earthquake event, date, recording station and epicenter. The processed data are presented after this, consisting of corrected and digitized ground motion time histories (acceleration, velocity, and displacement), Fourier spectral amplitudes, and response spectra for damping values of 0, 2, 5, 10, and 20%.

Only ground motion data were collected for this study. Those measurements in the mid-to-upper floors of buildings or on other types of structures such as dams were not considered, since their motions are affected by the dynamic characteristics of the structure. Data generated from earthquakes of unknown magnitude or unknown distance to the sources were also

excluded. However, recording stations with undefined site conditions were still retained (see Sec. 9.1.3.3).

Of the 741 records used here, 540 were obtained from CIT and were measured from earthquakes that occurred through the 1971 San Fernando event. These data have been used for previous ground motion studies (SW/AA, 1979; Werner and Ts'ao, 1977). The remaining 201 records are derived primarily from more recent earthquake events. The important feature of these new data is that many of the records were obtained at accelerograph stations located near the epicenter; i.e., of 121 sets of records used in this study that have the epicentral distance less than 20 km, 61 sets were obtained from the recent data source. Each set consists of three records, two horizontal and one vertical. A complete list of the recording stations where the data were collected is given by AA (App. B, AA, 1981).

9.1.3 GROUND MOTION PARAMETERS

9.1.3.1 Distance

There are several common definitions of distance for the study of ground motion attenuation:

- Epicentral distance
- Hypocentral distance
- Distance to the fault
- Distance to the center of energy release

When the recording station is far away from the earthquake source*, the measure of distance used does not alter the characteristics of the ground motion attenuation. However, when the

*The "earthquake source" here is used to indicate a point from where the distance is measured to the recording station. It can be the epicenter, the hypocenter, the center of energy release, etc.

station is in the near field, the measure of distance significantly affects the calculation of the attenuation rates. For example, for the October 15, 1979 El Centro earthquake, if the epicentral distance is used, Station No. 3 and Station No. 6 will be 28 and 26 km, respectively (see Fig. 8-3, in Chapter 8). If the fault distances are measured, Station No. 3 is about 2 km, whereas Station No. 6 is 17 km away. Also, the effect of distance on attenuation of ground motion depends on the rate of attenuation. For a slow attenuation rate, the effect of distance becomes less important. Only a rapid decrease in ground motion from the earthquake source requires a more careful definition of the distance.

The epicentral distance has been selected for use in this analysis, based on the following advantages:

- Epicentral distances are readily available.
- Except for stations very close to the fault, the fault distance often provides only a lower bound of the distance. The projected ground motion at the earthquake source using fault distance, therefore, results in a lower estimate.
- The estimate of hypocentral distance requires focal depth, which, so far, cannot be reliably measured.
- Extra computational effort is needed to estimate the distance to the center of energy release.

9.1.3.2 Specific Earthquake Events

More than half the records for this study were measured during two significant earthquake events – the San Fernando earthquake of February 9, 1971 (294 records) and the Imperial

Valley earthquake of October 15, 1979 (85 records). The Santa Barbara earthquake, although it produced only a few records, was selected for comparison. For convenience, these earthquakes are classified as:

- Type 1: All earthquakes
- Type 2: San Fernando earthquake of Feb. 9, 1971
- Type 3: Imperial Valley earthquake of Oct. 15, 1979
- Type 4: Santa Barbara earthquake of Aug. 13, 1978

Data from these events were used to perform additional statistical analyses to identify distinguishing ground response characteristics for these events and to illustrate how the large amount of data from these two events can influence the composite ground response characteristics obtained from this study.

9.1.3.3 Site Conditions

The procedures used to represent the site conditions are similar to those used in SW/AA (1979). These procedures are based on the following site classifications, based on the subsurface material properties within a few hundred feet of the ground surface.

- Rock sites – 20 ft or less of overburden that overlies rock-like materials.
- Stiff sites – stiff clays and/or dense sands whose depth to rock-like materials ranges from about 20 to 250 ft.
- Deep soil sites – stiff clays and/or dense sands whose depth to rock-like material is approximately 250 ft or greater.

Of the 247 sets of records studied, 66 sets of records fall into the deep soil site category, 34 correspond to stiff sites, and 15 represent rock sites. The remaining 132 sets of records do not have enough geological data to classify

the site condition. Since these remaining records with unknown site conditions possess other valuable information such as distance, magnitude, and specific earthquake events, etc., it is desirable to retain them in the statistical study and to represent their site conditions in an approximate way. This was done by assuming that each site with unknown conditions is split into the above three classifications, by using weighting factors defined in accordance with the distribution of known sites. On this basis, each unknown site is represented as 20/36 of the deep site, 12/36 of a stiff site and 4/36 of a rock site (which represent values close to the actual distribution of known sites). However, some of these sites may not have completely unknown conditions; instead, they may be judged to be between two different site conditions. For this case, judgment is used to assign the weighting factors. For example, weighting factors of 1/2 for deep site and 1/2 for stiff site may be assigned if the actual site is judged to be midway between the deep and stiff soil classifications, or may lean greater toward either the deep or the stiff classification if the properties tend toward one or the other.

9.1.3.4 Earthquake Magnitude

The size of the earthquake is generally represented here using the local magnitudes, although for certain records surface magnitude was used. The local magnitude is not as desirable a parameter as, say, seismic moment for representing the earthquake size; however, it is the only one readily available for use in this manner, and therefore has been adopted. The resulting relationship between the local magnitude and the ground motions is established on the premise that the ground motion amplitude increases with magnitude at a decreasing rate (CIT, 1969-75).

9.1.4 STATISTICAL ANALYSIS PROCEDURE

The following step-by-step procedure is used in carrying out the analysis for the spectral amplitude. Additional details for each step of the approach are given in AA (App. A, AA, 1981).

Step 1: Form the Basic Regression Equation

The independent variables in the regression analysis are identified and incorporated into the following multiple regression equation:

$$v = f(x_1, x_2, \dots, x_m) \quad (9-1)$$

$$= a_1 x_1 + a_2 x_2 + \dots + a_m x_m$$

where y is the dependent variable (e.g., spectral amplitude)*, x_i ($i = 1, \dots, m$) are functions of the independent variables (e.g., magnitude, distance, etc., or combinations thereof) and a_i ($i = 1, \dots, m$) are the unknown regression coefficients. The identification of the x_i is based on engineering judgment and on guidance from appropriate theoretical results that interrelate x_i and y .

Step 2: Determine the Regression Coefficients

The regression coefficients, a_i ($i = 1, \dots, m$) in Equation 9-1, are determined using a standard least-squares technique for fitting the equation.

Step 3: Carry Out Step-Down Algorithm

A step-down algorithm assures that only statistically significant terms are retained on the right side of Equation 9-1. The first step in this algorithm uses the F-test to eliminate the most statistically insignificant of the original terms, if the F-value for the term falls below the 95th percentile of the F-distribution.

* y may also be a transformation of the dependent variable. For example, for several of the ground motion parameters studied, $y = \ln y$ was used, where y is the dependent variable.

Then, a new set of regression coefficients is determined for the remaining terms (Step 2) and the elimination process is repeated. This algorithm is completed when all remaining terms have an F-value that falls above the 95th percentile.

Step 4: Further Investigate Particularly Significant Variables

The effects of variables shown from Step 3 to be particularly significant are investigated further by defining cross products, higher order terms, etc., involving these variables and repeating Steps 1 to 3 to determine if these additional terms are significant.

Step 5: Evaluate the Regression Equations Based on Near Field Data

Steps 1 to 4 are repeated for several regression equations. These equations are then compared to each other. The criterion for selecting the final regression equation is based on goodness of fit of the data to these equations. These equations are also compared with near field data as another judging factor.

Step 6: Compute Data Fit Parameters and Identify Outliers

Once the regression equation is determined by Steps 1 to 4, the standard deviation, coefficient of variation, and multiple correlation coefficient are computed. In addition, outliers are identified as ground motion records whose residual exceeds 3σ , where σ is the standard deviation of the spectral amplitude (i.e., y in Equation 9-1).

Step 7: Construct Statistical Response Spectrum

For the prescribed magnitude, distance, site conditions, and direction (horizontal or vertical component) plugged into each of the 32 regression equations, 32 spectral amplitudes corresponding to 32 frequencies are computed. The statistical response spectrum is obtained by linearly connecting these 32 points.

9.1.5 REGRESSION EQUATION

The final form of the regression equation is based on the use of the above procedures to assess several possible regression equation forms. This assessment – carried out at frequencies of 25.0 Hz, 1.43 Hz, and 0.067 Hz – should define the regression equation form that provides the best fit to the data at these frequencies. The selected form can then be implemented at the remaining 29 frequencies, as shown in Table 9-1. The outliers at these frequencies are listed in Table 9-2.

An important aspect of this effort is to define the form of the terms involving the epicentral distance, R . From wave theory, the following two models were considered to represent distance effects:

$$\text{Model 1: } (R + \Delta) - a$$

$$\text{Model 2: } (R^2 + \Delta)^{-a}$$

where " a " was obtained from the regression analysis and Δ was determined so as to minimize the standard deviation (at frequencies of 25.0 Hz, 1.43 Hz, and 0.067 Hz). This resulted in Δ values of 7.0 for Model 1 and 225.0 for Model 2 (see Table 9-3a), each of which resulted in very similar standard deviations at the three frequencies. The two models were also used to estimate horizontal spectral velocities at 25 hz for the case where $R = 0$ and for magnitudes of 5, 6, and 7. Model 2 resulted in only slightly lower spectral velocities (Table 9-3b) although the values of spectral velocity indicated by each model appear reasonable. Therefore, it is seen from Tables 9-3a and b that there is little difference between the two models to represent distance effects. So, Model 2 was used to represent distance effects, although either model would have produced comparable results.

TABLE 9-1. REGRESSION EQUATIONS FOR RESPONSE SPECTRAL AMPLITUDES
(Continued next page)

Regression Equation:

$$\begin{aligned} \ln[S_v(f)] = & A_1(f) + \frac{A_2(f)}{M} + \frac{A_3(f)}{M^2} + A_4(f) \ln(R^2 + 225) \\ & + A_5(f) \cdot V + A_6(f) \cdot S + A_7(f) \cdot E + [A_8(f) \cdot B + A_9(f) \cdot S \\ & + A_{10}(f) \cdot D + A_{11}(f) \cdot K] \cdot \ln(R^2 + 225) \end{aligned}$$

where

$$\begin{aligned} S_v(f) &= \text{Pseudovelocity (ips) for frequency } f \text{ and damping ratio} = 0.05 \\ M &= \text{Local magnitude} \\ R &= \text{Epicentral distance, km} \\ V &= \begin{cases} 1 & \text{(if vertical component)} \\ 0 & \text{(if horizontal component)} \end{cases} \\ S &= \begin{cases} 1 & \text{(if San Fernando earthquake event, Type 2)} \\ 0.4008 & \text{(if all earthquakes considered, Type 1)} \\ 0 & \text{(if earthquake event other than San Fernando)} \end{cases} \\ E &= \begin{cases} 1 & \text{(if Imperial Valley earthquake event, Type 3)} \\ 0.1147 & \text{(if all earthquakes considered, Type 1)} \\ 0 & \text{(if earthquake event other than Imperial Valley)} \end{cases} \\ D &= \begin{cases} 1 & \text{(if deep soil site)} \\ 0 & \text{(if non-deep soil site)} \end{cases} \\ B &= \begin{cases} 0.0135 & \text{(if all earthquakes considered, Type 1)} \\ 1 & \text{(if Santa Barbara earthquake event, Type 4)} \\ 0 & \text{(if earthquake event other than Santa Barbara)} \end{cases} \\ K &= \begin{cases} 1 & \text{(if rock site)} \\ 0 & \text{(if non-rock site)} \end{cases} \end{aligned}$$

TABLE 9-1. REGRESSION EQUATIONS FOR RESPONSE SPECTRAL AMPLITUDES
(Concluded)

Frequency, Hz	A ₁	A ₂	A ₃	A ₄	A ₅	A ₆	A ₇	A ₈	A ₉	A ₁₀	A ₁₁
25.000	9.3587	-56.0330	90.6894	-0.5583	-0.5848	1.7553	0.4056	0.1532	-0.2226	0.0258	0
20.000	9.9152	-57.9717	95.5338	-0.5710	-0.5275	1.7980	0.5337	0.1593	-0.2287	0.0238	0
16.667	10.1188	-57.9103	96.1068	-0.5671	-0.4677	2.0161	0.5845	0.1505	-0.2560	0.0215	0
14.300	10.7115	-61.5396	105.7543	-0.5728	-0.4138	2.0817	0.5998	0.1500	-0.2602	0.0209	0
11.800	11.0913	-61.7219	106.8710	-0.5804	-0.4168	2.0152	0.5566	0.1381	-0.2477	0.0231	0
10.000	12.1831	-68.0675	120.8405	-0.6008	-0.4543	1.9603	0.5355	0.1519	-0.2374	0.0223	0
8.333	12.1782	-65.2703	114.2872	-0.5991	-0.4866	1.9211	0.5251	0.1598	-0.2265	0.0201	0
6.667	11.6724	-57.1999	93.9439	-0.5923	-0.5798	1.7159	0.4163	0.1681	-0.2028	0.0262	0
5.000	11.7462	-55.0226	86.1800	-0.5718	-0.6797	1.4620	0.3333	0.1537	-0.1683	0.0267	0
4.167	11.9387	-56.2211	90.4628	-0.5594	-0.7668	1.5709	0.2581	0.1392	-0.1854	0.0268	0
3.560	12.4509	-58.6176	91.9963	-0.5610	-0.8385	1.4151	0.1760	0.1838	-0.1639	0.0291	-0.0081
3.030	12.8265	-60.8516	94.7572	-0.5485	-0.8886	1.2561	0.1598	0.2024	-0.1467	0.0262	-0.0270
2.500	12.1846	-57.1099	84.1093	-0.5051	-0.9226	2.1735	0.1199	0.2133	-0.2581	0.0352	-0.0309
2.000	12.7791	-63.7225	96.9557	-0.4777	-0.9087	1.8091	0.1550	0.2623	-0.2218	0.0360	-0.0439
1.667	12.8730	-64.1549	93.2728	-0.4639	-0.9109	2.1186	0.1397	0.3069	-0.2582	0.0366	-0.0608
1.430	12.7952	-63.3084	87.4723	-0.4433	-0.8858	2.2135	0.1559	0.3306	-0.2708	0.0250	-0.0668
1.180	13.0622	-64.7837	86.7779	-0.4420	-0.8758	2.8459	0.1376	0.3127	-0.3415	0.0273	-0.0775
1.000	12.6163	-62.4477	81.7139	-0.4208	-0.8470	3.3917	0.2288	0.3048	-0.3981	0.0286	-0.0779
0.833	12.4468	-61.3281	78.0855	-0.4190	-0.8153	3.7564	0.3075	0.3093	-0.4353	0.0333	-0.0843
0.667	11.7627	-58.9093	70.4738	-0.3731	-0.7725	4.2744	0.4546	0.3417	-0.4954	0.0452	-0.0837
0.500	11.8984	-65.6188	90.0407	-0.3244	-0.6930	4.5795	0.6571	0.2879	-0.5303	0.0339	-0.0922
0.416	12.9964	-79.5889	129.7931	-0.3169	-0.6556	4.5380	0.7499	0.2376	-0.5033	0.0284	-0.1014
0.356	12.9846	-81.6581	136.5479	-0.3031	-0.6368	4.4578	0.7927	0.1916	-0.4812	0.0328	-0.1063
0.303	13.1246	-85.0929	145.5206	-0.2914	-0.5663	4.3525	0.8965	0.1509	-0.4461	0.0323	-0.1069
0.250	12.8288	-85.4908	148.6255	-0.2641	-0.5819	4.4040	1.0170	0.1224	-0.4371	0.0320	-0.1032
0.200	12.1346	-83.2602	148.0086	-0.2338	-0.6541	4.1028	1.1222	0.0907	-0.3896	0.0336	-0.0944
0.167	11.6157	-79.3470	138.1443	-0.2185	-0.7266	3.0867	0.9676	0.0690	-0.2768	0.0273	-0.0756
0.143	11.9944	-83.9884	151.1591	-0.2202	-0.7226	2.1590	0.7659	0.0553	-0.1815	0.0209	-0.0674
0.118	11.5303	-81.8288	147.7622	-0.2025	-0.6402	2.0558	0.4682	0.0357	-0.2052	0.0143	-0.0602
0.100	10.4120	-73.0714	126.9985	-0.1825	-0.6039	2.4507	0.2873	0.0200	-0.2841	0.0135	-0.0535
0.083	9.7789	-69.8840	121.7356	-0.1655	-0.5400	2.5972	0.1134	0.0098	-0.3217	0.0114	-0.0543
0.067	8.9568	-62.2183	102.5127	-0.1964	-0.5002	2.5892	0	0	-0.3209	0.0208	-0.0514

TABLE 9-2. OUTLIERS FROM REGRESSION ANALYSES AT THREE SELECTED FREQUENCIES

Freq Hz	Accelerograph Station		Earthquake Event			Site Condition	Horizontal or Vertical Component	Pseudo- velocity, ips	Deviation of $\ln S_v(f)$
	Location	Epicentral Distance, km	Location	Date	Mag				
25	Irrig Dist	23 6	Imperial Valley	01/13/53	5 5	Deep	H	0 019	-3 7 σ
	Shelter Cove No 2	34 0	Shelter Cove	05/06/75	4 0	Undefined	H	0 551	3 3 σ
	L B Public Util Bldg	6 2	Torrance-Gardena	11/14/41	5 4	Undefined	V	0 024	-3 2 σ
	S Pac Bldg	16 2	San Francisco	03/22/54	3 8	Undefined	V	0 005	-3 2 σ
	S Pac Bldg	16 2	San Francisco	03/22/57	3 8	Undefined	H	0 007	-3 5 σ
	McCabe School No 11	27 0	El Centro	10/15/79	6 6	Deep	V	5 315	3 7 σ
	S Pac Bldg	18 3	San Francisco	03/22/57	4 0	Undefined	H	0 007	-3 6 σ
1 43	Irrig Dist	23 6	Imperial Valley	01/13/53	5 5	Deep	H	0 452	-3 3 σ
	Ft Tejon, CA	68 5	San Fernando	02/09/71	6 4	Stiff	H	0 329	-3 4 σ
	Nicaragua	19 0	Managua	01/02/72	4 1	Undefined	H	7 189	3 3 σ
	Nicaragua	19 0	Managua	01/05/72	4 2	Undefined	H	7 890	3 3 σ
0 067	Shelter Cove No 2	60 0	Shelter Cove	06/07/75	5 7	Undefined	H	0 063	-3 4 σ
	L B Public Util Bldg	27 2	Long Beach	03/10/33	6 3	Undefined	V	8 600	3 3 σ

TABLE 9-3. COMPARISON OF ALTERNATIVE MODELS FOR REPRESENTING EPICENTRAL DISTANCE EFFECTS*

(a) Standard Deviation

Frequency Hz	Model 1 $(R + 7.0)^{-a}$	Model 2 $(R^2 + 225.0)^{-a}$
25.0	0.706	0.699
1.43	0.666	0.667
0.067	0.697	0.695

(b) Horizontal Spectral Velocities at 25 Hz and $R = 0$

Magnitude	Spectral Velocity, ips	
	Model 1 $(R + 7.0)^{-a}$	Model 2 $(R^2 + 225.0)^{-a}$
5	0.287	0.246
6	0.585	0.505
7	1.099	0.963

*Conditions Considered in Comparisons

Earthquake Event Type = 3
(Imperial Valley earthquake of Oct 15, 1979)
Site Condition = Stiff

9.2 STATISTICAL ANALYSIS RESULTS

9.2.1 COMPARISONS WITH RECORDED DATA

The computed response spectra (CRS) derived from this study represent the general behavior of the earthquake ground motion. The goodness-of-fit of the CRS to the real data is shown by the standard deviations and the correlation coefficients. A smaller standard deviation represents a better fit of the regression equation to the data, and a higher value of correlation coefficient means the parameters under consideration are closer related than the low value of correlation coefficient.

Shown in Table 9-4 are the 32 sets of standard deviations and correlation coefficients. This table indicates that the CRS have a fairly uniform scatter throughout the frequency range

TABLE 9-4. STANDARD DEVIATIONS AND CORRELATION COEFFICIENTS OF THE COMPUTED SPECTRAL AMPLITUDES

Sequence Number	Frequency, Hz	Standard Deviation, in./sec	Correlation Coefficient
1	25.0	0.699	0.762
2	20.0	0.720	0.762
3	16.67	0.752	0.748
4	14.3	0.766	0.743
5	11.8	0.793	0.729
6	10.0	0.800	0.735
7	6.33	0.791	0.739
8	6.67	0.765	0.744
9	5.00	0.729	0.754
10	4.16	0.710	0.761
11	3.56	0.693	0.775
12	3.03	0.690	0.778
13	2.50	0.682	0.785
14	2.00	0.644	0.800
15	1.67	0.638	0.811
16	1.43	0.667	0.799
17	1.18	0.680	0.805
18	1.00	0.690	0.805
19	0.833	0.697	0.810
20	0.667	0.686	0.821
21	0.500	0.706	0.812
22	0.416	0.750	0.798
23	0.356	0.752	0.799
24	0.303	0.769	0.802
25	0.250	0.775	0.808
26	0.200	0.764	0.811
27	0.167	0.748	0.799
28	0.143	0.732	0.778
29	0.118	0.704	0.741
30	0.100	0.703	0.706
31	0.083	0.709	0.666
32	0.067	0.695	0.659

of interest (i.e., the differences in standard deviations among different frequencies are small). The smallest standard deviations are observed in the middle frequency range somewhere from 0.6 Hz to 3 Hz. With the exception of the lowest frequencies, the correlation coefficients are higher than 0.7; the highest correlation coefficients are found in the middle frequency range.

Another way of comparing the CRS with recorded data is by visual inspection, by plotting a ground response spectra from actual measured records versus the CRS with the same parametric conditions as those of the measured records (i.e., same magnitude, epicentral distance, site conditions, etc.). Because of the random nature of the ground motion, it is expected that each individual response spectrum at a specified frequency will fluctuate around the mean values of the CRS, and that for a given frequency, 68% of the spectral amplitudes of the measured records will fall within one standard deviation of the mean.

Eight sets of real ground motion spectra along with the CRS curves were used in this comparison (see AA, 1981). The records selected for comparison cover a range of magnitudes, distances, site conditions, and earthquake events. Figure 9-1 provides an example of these comparisons, showing the recorded spectra for Bond's Corner (El Centro earthquake, 1979) and the CRS. The 84% (mean plus one standard deviation) and 16% (mean minus one standard deviation) of the CRS curves are also presented in this figure in order to compare each record relative to the CRS and its deviations. A summary of the evaluation of these selected records in relation to the CRS curves is given in Table 9-5.

9.2.2 PARAMETRIC STUDIES

Several parametric studies were used extensively in this analysis (AA, 1981), in

(a) Horizontal

(b) Vertical

FIGURE 9-1. COMPARISON BETWEEN COMPUTED AND RECORDED GROUND RESPONSE SPECTRA (Bond's Corner, Imperial Valley Earthquake, 1979)

TABLE 9-5. COMPARISONS OF REAL EARTHQUAKE RECORDS WITH THE CRS

Event	Location	Mag.	Dist. (km)	Cond.	Goodness of fit with respect to CRS*		
					Low Frequency Range	Middle Frequency Range	High Frequency Range
San Fernando 1971	Pacoima Dam	6.4	9.1	Rock	(H) Good (V) Above 84%	Above 84% Above 84%	Around 84% Above 84%
San Fernando 1971	2500 Wilshire Blvd.	6.4	40.7	Stiff	(H) Good (V) Good	Good Good	Good Good
Imperial Valley 1979	Bond's Corner	6.6	15.0	Deep	(H) Good (V) Around 16%	Around 84% Good	Good Good
Imperial Valley 1979	Station #12	6.6	30.0	Deep	(H) Good (V) Good	Good Around 16%	Around 16% Good
Kern County 1952	Caltech Atheneum	7.7	126.0	Stiff	(H) Above 16% (V) Good	Good Good	Above 16% Good
Parkfield 1966	Temblor Number 2	6.0	31.0	Rock	(H) Above 84% (V) Above 84%	Above 84% Around 84%	Around 84% Around 84%
Coyote 1979	Gilroy Number 1	5.7	15.7	Deep	(H) Below 16% (V) Above 16%	Around 16% Below 16%	Good Good
Shelter Cove 1975	Ferndale City Hall	5.7	5.0	Deep	(H) Around 16% (V) Around 16%	Good Above 16%	Good Around 16%

*The descriptions given here refer to the closeness of the computed response spectra (CRS) to the recorded response spectra, sometimes in terms of its relation to the 84th percentile or 16th percentile spectra (see Fig. 9-1).

order to compare CRS curves for different parameters. The parameters considered were: (1) specific earthquake events, (2) earthquake magnitudes, (3) site conditions, (4) vertical and horizontal components, and (5) different distance definition.

9.2.3 COMPARISON WITH REGULATORY GUIDE 160

The Regulatory Guide (RG) 1.60, based on the procedures developed by Newmark, Blume, and Kapur (1973), is widely used for defining seismic input criteria for nuclear power plants. It consists of a set of amplification factors for various damping ratios at five discrete frequencies. To develop the actual design spectra, the criteria peak horizontal acceleration is multiplied by these amplification factors. The resulting spectra are a series of straight lines connecting these resulting values. It is noted that the data base used to develop RG 1.60 is much less than that used in the present study, and the RG 1.60 spectrum shapes do not change with

varying magnitude, distance, and site conditions.

Since the computed response spectra (CRS) obtained in this study involve several parameters, each combination of parameters results in a different level of peak horizontal acceleration. A reasonable level of acceleration used in RG 1.60 to compare with this study is the 50th percentile peak horizontal acceleration. For all the comparisons investigated here, Type 1 earthquake events are used (see Sec. 9.1.3.2 above). Two sets of comparisons are made: one corresponding to near field spectra and the other corresponding to midfield spectra. Variation of magnitudes and different site conditions are considered in each set of comparisons.

9.2.3.1 Midfield Spectra vs. RG 1.60

The first set of comparison with RG 1.60 is shown in Figures 9-2 through 9-5. In these

EPICENTRAL DISTANCE = 50 km
 SITE CONDITION = ROCK
 EARTHQUAKE EVENT = TYPE 1
 MAGNITUDE = 6.5
 DAMPING RATIO = 0.05

(a) Horizontal component

(b) Vertical component

FIGURE 9-2. COMPARISON OF MIDFIELD SPECTRA (ROCK SITE) WITH RG 1.60

EPICENTRAL DISTANCE = 50 km
 SITE CONDITION = DEEP
 EARTHQUAKE EVENT = TYPE 1
 MAGNITUDE = 6.5
 DAMPING RATIO = 0.05

(a) Horizontal component

(b) Vertical component

FIGURE 9-3. COMPARISON OF MIDFIELD SPECTRA (DEEP SOILS SITE) WITH RG 1.60

EPICENTRAL DISTANCE = 50 km
 SITE CONDITION = STIFF
 EARTHQUAKE EVENT = TYPE 1
 MAGNITUDE = 6.5
 DAMPING RATIO = 0.05

(a) Horizontal component

(b) Vertical component

FIGURE 9-4. COMPARISON OF MIDFIELD SPECTRA (STIFF SOILS SITE) WITH RG 1.60

EPICENTRAL DISTANCE = 50 km
 SITE CONDITION = STIFF
 EARTHQUAKE EVENT = TYPE 1
 MAGNITUDE = 8.0
 DAMPING RATIO = 0.05

(a) Horizontal component

(b) Vertical component

FIGURE 9-5. COMPARISON OF MIDFIELD SPECTRA ($M = 8.0$) WITH RG 1.60

figures, the peak acceleration of RG 1.60 spectra and the 50th percentile CRS spectra are made equal. CRS for different site conditions, magnitudes, and ground motion components are plotted for an epicentral distance of 50 km. The comparisons lead to the following observations:

- a. Overall, the RG 1.60 provides a more conservative response spectrum than CRS. Also, the RG 1.60 does not possess a uniform statistical confidence level, i.e., it does not have the same confidence level at different frequencies.
- b. Almost all the vertical components derived from RG 1.60 exceed the 84th percentile CRS, although the criteria peak horizontal acceleration is based on 50th percentile of the CRS. Therefore, the vertical spectra estimated for the RG 1.60 is more conservative than that of the horizontal spectra.
- c. Among the different site conditions, the RG 1.60 exceeds the CRS for the rock site by a substantial amount in the frequency ranges of 2 Hz and lower. For the horizontal component of the deep site and stiff site, the RG 1.60 falls between 50th percentile and 84th percentile CRS.
- d. Even at the magnitude level of 8.0, the RG 1.60 horizontal spectrum is slightly higher than that of 50th percentile CRS, but the vertical spectrum is closer to 84th percentile CRS.

9.2.3.2 Near Field Spectra vs. RG 1.60

Comparisons of the CRS with RG 1.60 for the near field with epicentral distance of 10 km are shown in Figures 9-6 through 9-9. In

these figures, the peak accelerations of RG 1.60 spectra and the 50th percentile CRS spectra are made equal. These comparisons lead to the following observations:

- a. The RG 1.60 provides a more conservative response spectrum than CRS. The difference is even more pronounced in the near field than in the midfield.
- b. The vertical component of RG 1.60 exceeds all the 84th percentile of the CRS. For the rock site with magnitude 6.5, the RG 1.60 spectral amplitudes are more than two standard deviations greater than the 50th percentile CRS in the frequency range of 3.5 Hz and lower.
- c. Except as noted above, the differences between RG 1.60 and CRS are similar to those for the midfield case.

9.2.4 RECOMMENDATION FOR FUTURE STUDIES

Numerous regression analyses have been carried out in recent years using ground motion data. Fairly good agreement has been obtained regarding the prediction of the ground motion in the mid to far field regions, and now, due to the availability of near field data, more attention is focused on the study of near field ground motion.

Until now, almost all data used in the analyses were obtained directly from various sources (CIT, USGS, etc.) without modification. The quality of the data can be improved by these considerations:

- a. The instrumentation station records the ground motion in the preset directions; these directions should be transformed into the direction of principal wave motions so that the

EPICENTRAL DISTANCE = 10 km
 SITE CONDITION = ROCK
 EARTHQUAKE EVENT = TYPE 1
 MAGNITUDE = 6.5
 DAMPING RATIO = 0.05

(a) Horizontal component

(b) Vertical component

FIGURE 9-6. COMPARISON OF NEAR FIELD SPECTRA (ROCK SITE) WITH RG 1.60

EPICENTRAL DISTANCE = 10 km
 SITE CONDITION = DEEP
 EARTHQUAKE EVENT = TYPE 1
 MAGNITUDE = 6.5
 DAMPING RATIO = 0.05

(a) Horizontal component

(b) Vertical component

FIGURE 9-7. COMPARISON OF NEAR FIELD SPECTRA (DEEP SOILS SITE) WITH RG 1.60

(a) Horizontal component

(b) Vertical component

FIGURE 9-8. COMPARISON OF NEAR FIELD SPECTRA (STIFF SOILS SITE) WITH RG 1.60

(a) Horizontal component

(b) Vertical component

FIGURE 9-9. COMPARISON OF NEAR FIELD SPECTRA ($M = 8.0$) WITH RG 1.60

correct ground motion amplitudes and directions are used.

- b. Many recording stations are located in the buildings. The ground motions recorded in these stations may be influenced by the building responses.

Results of the tests of building structures may be employed to filter out the building effects on the records, so that the resulting ground motion records represent true free field motions.

CHAPTER 10

CONTRIBUTIONS OF BODY AND SURFACE WAVES

10.1 INTRODUCTION

It is hypothesized that separating strong motion acceleration records into components of body and surface wave motion would permit the development of better correlation relationships between peak ground motion (i.e., acceleration, velocity, and displacement) and earthquake magnitude, source distance, and site subsurface conditions. Also, a better knowledge of the relative contribution of body and surface wave motion to the total motion would permit the development of better soil/structure interaction analysis procedures. There is, however, no procedure available that permits separating earthquake strong motion records into body and surface wave components of motion, nor could one be readily devised within the scope of this investigation.

The results presented here add up to a preliminary investigation in which the problems associated with a study to determine the relative contribution of body and surface wave motions to peak earthquake ground motion are identified and a feasible approach to a broader based investigation can be defined. A qualitative estimate of the relative contribution of body and surface wave motions to the peak ground motion recorded at 11 stations during the 1971 San Fernando earthquake is provided here.

10.2 COLLECTION OF DATA

10.2.1 BACKGROUND

It is generally assumed in soil/structure interaction analyses of nuclear power plant structures that the design earthquake ground motion consists of nearly vertically propagating

body waves (P and S waves, that is, dilatational and shear waves). Studies indicate that strong surface wave motions (Rayleigh and Love waves) may also be present, which requires a more complex soil/structure interaction modeling procedure than currently used (AA, 1979). Studies by Hanks (1975) and by Drake and Mal (1972) have contributed important information on surface wave motions generated by the 1971 San Fernando earthquake.

Hanks investigated the presence of surface wave motions in strong motion records monitored along four major azimuths during the 1971 San Fernando earthquake. These azimuths are indicated in Figure 10-1 as Profiles 1 through 4. Also shown in this figure is the radiation pattern of 8-sec Rayleigh waves based on the source model of Alewine (1974), which consists of four spatially and temporally separated point sources, arranged such that fault rupture is initiated in the hypocentral region and then propagates to the south at an effective rupture velocity of 3.2 km/sec. Hanks worked from displacement time histories computed from acceleration records monitored along each of the four azimuth ranges. The results of the study indicate that the most intense surface wave motions developed along the north-south azimuth (Profile 4), which is in agreement with the Rayleigh wave radiation pattern. For this reason, acceleration records from fifteen strong motion recording stations along this azimuth were selected for study here (see Table 10-1). The first eleven stations in Table 10-1 are those considered by Hanks. An additional four stations (also listed in the table) are considered in this study.

The 11 stations considered by Hanks are associated with a variety of site conditions (AA,

FIGURE 10-1. RADIATION PATTERN OF RAYLEIGH WAVES AND AZIMUTH RANGES INVESTIGATED BY HANKS, 1975

1980c). Two stations are on crystalline rock in regions of rapidly changing topography, four are located on deep soils that are probably saturated at depth and of low to medium density, several are located on soft sedimentary rock, or on shallow soils overlying sedimentary rock, in regions where the basement rock is either rapidly decreasing or rapidly increasing, and three are located on alluvial or terrace deposits of undetermined depth near the south slope of the mountains adjacent to a significant fault zone. There are, therefore, a variety of factors that could have influenced the wave forms present in the records monitored at these stations during the earthquake.

In order to minimize long-period processing errors, Hanks first examined the displacement time histories of four separate areas in which the recording stations were so closely spaced in each area that the transient time of long period motion across each station cluster was negligibly small compared to the wave length of the long period motion. Average displacement time

histories for the stations in each cluster were then computed. One area that is significant to the profile extended for a distance of only four blocks from 3345 to 3710 Wilshire Boulevard and included five recording stations that were all 40 km from the epicenter. The records were synchronized in time by assigning the same time (6.4 sec) to the first arrival of a high frequency shear wave on each record. The average vertical, radial, and transverse time histories computed by Hanks for the five stations are provided in Figure 10-2. The trajectory of particle motion in the vertical radial plane that was constructed from the average vertical and average radial time histories is also provided in this figure, with the letters parameterizing time.

Hanks interprets the early arriving longer period motion between B and E in the trajectory diagram in Figure 10-2 to be long period Rayleigh waves because of the elliptical retrograde motion indicated in the vertical radial plane. Shorter period motion between F and J

FIGURE 10-2. AVERAGE DISPLACEMENT TIME HISTORIES AND PARTICLE MOTION TRAJECTORY DIAGRAM FOR FIVE STATIONS ON WILSHIRE BLVD. (Hanks, 1975)

is indicated on both the vertical and radial time histories but not on the transverse time history. Since the Rayleigh waves appear on the vertical radial plane and the Love waves appear on the transverse plane, Hanks interprets these motions to be shorter period (perhaps higher mode) Rayleigh waves. It will be noted that the particle motion trajectory within this time interval (E to J) is quite erratic and is not always retrograde elliptical. The elongation of the ellipses in radial direction is indicated to result from the low seismic

velocity of the surface layer relative to the seismic velocity of the substratum.

Particle motion trajectory diagrams are not provided by Hanks for stations along the four major azimuths investigated. Vertical, radial, and transverse displacement time histories are provided, and conclusions appear to have been based upon the displacement time histories. AA (1980c) provides the vertical and radial displacement time histories given by Hanks for the stations extending north-south across the San

Fernando valley (Profile 4), and for stations extending across the Santa Monica Mountains and the Los Angeles Basin to the Palos Verdes Hills along the same profile.

Drake and Mal (1972) also indicate the importance of the surface wave contribution to the ground motion recorded at stations in the San Fernando Valley during the 1971 earthquake. This study is based on a two-dimensional finite element model of a north-south profile of the San Fernando Valley from which surface wave (Love and Rayleigh) mode shapes were extracted at different stations along the profile. The profile considered is the same as Profile 4 considered by Hanks.

This study is important to engineers because it demonstrates the effect of layered rock and soil media of increasing density with depth on the shape of the surface wave, and also demonstrates the depth to which surface wave effects frequently extend. AA (1980c) provides details of this study. The study indicates that surface wave motions extend several thousand feet below the ground surface

and do not attenuate significantly in the top 50 to 100 ft, which is the depth of greatest interest in soil/structure interaction analyses. Even though an attenuation in amplitude is normally expected with distance, larger amplitude Love and Rayleigh wave motions are indicated as the seismic waves advance for the subsurface conditions considered. Drake and Mal also note that while the fundamental Rayleigh wave particle motion at the surface is retrograde elliptical, the higher modes display prograde motion. It is noted in the trajectory diagram in Figure 10-2 that the higher frequency motion after time E is a mixture of motions that could represent higher mode prograde Rayleigh wave motions superimposed on the retrograde fundamental Rayleigh wave motion. Trajectory diagrams given later will exhibit similar complex motion.

10.2.2 STRONG MOTION RECORDING STATION DATA

Data were also collected for four additional stations (Stations 12 through 15 in Table 10-1)

TABLE 10-1. RECORDING STATION DATA: PROFILE 4

Station Number	Address and Location	Epicentral Distance	Azimuth, Degrees
1	Pacoima Dam	9	180
2	Los Angeles, 8244 Orion, 1st floor	22	199
3	Los Angeles, 15107 Vanowen, basement	24	197
4	Los Angeles, 15250 Ventura Blvd., basement	29	194
5	Los Angeles, 3838 Lankershim, basement	31	170
6	Los Angeles, 7080 Hollywood, basement	35	168
7	Los Angeles, Hollywood Storage Bldg., basement	37	168
8*	Los Angeles, 5900 Wilshire, "B" Parking level	39	172
9	Los Angeles, 8639 Lincoln, basement	50	185
10	Los Angeles, 5260 Century, 1st floor	52	174
11	Palos Verdes Estates, 2516 Via Tejon	68	176
12*	Los Angeles, 14724 Ventura Blvd.	29	191
13*	Los Angeles, 15910 Ventura Blvd.	29	196
14	Los Angeles, Griffith Park Observatory	34	163
15*	Los Angeles, 120 N. Robertson	37	175

*Not included in Table 10-2, due to limited scope of this study.

in order to observe the effect of local variations in site conditions on the recorded wave forms. AA (1980c) provides a brief description for each station of the building housing the accelerograph, the location of the accelerograph within the building, the site subsurface conditions to the extent known, acceleration, velocity, and displacement time histories and response spectra for the 1971 San Fernando earthquake. The time histories and response spectra obtained from the recorded wave forms at these stations are processed and published by the California Institute of Technology (CIT, 1973). These records have also been provided for convenience in the appendix of AA (1980c).

10.3 ANALYSIS OF DATA

At the start of this study it was anticipated that some means could be devised within the time and funding constraints that would permit separating acceleration records into components of body and surface wave motion. Unfortunately, no procedure could be found in the technical literature nor could one be devised that would accomplish this objective. Since a satisfactory procedure of separating the acceleration records could not be devised, only a qualitative estimate of the relative contribution of body and surface wave motions to the peak ground motions has been provided in this pilot study.

10.3.1 QUALITATIVE EVALUATION OF GROUND MOTION RECORDS

The estimate given here of the relative contribution of body and surface wave motions to the peak ground motions recorded at a series of selected stations during the 1971 San Fernando earthquake is termed "qualitative" since the wave contributions for each station have been based on a visual study of the ground

motion records, response spectra, and a particle motion trajectory diagram for the station. In cases where the wave identification is correct, the estimates should be correct to within a precision of about 5%. In some cases, which are noted, the wave identification is uncertain and may be in error. For those cases, the error may be as great as 30%.

Only the ground motions recorded along the 4th azimuth (see Fig. 10-1) are considered. In order to limit the study, only motions in the vertical radial plane were considered. Ultimately the study should be extended to include stations along an azimuth not experiencing strong surface wave motions, and motions should be considered in the transverse as well as in the radial planes along both azimuths.

Horizontal and vertical time motion records and response spectra were assembled for the fifteen stations (see Table 10-1), and are included in AA (1980c). The horizontal components of motion and the horizontal response spectra are in most cases radial, or near radial, components (i.e., north-south). However, at a few stations the accelerograph sensors were not oriented north-south and east-west, but were oriented from 7 to 45 deg out of the radial and tangential planes.

The first step in analyzing the data was to construct particle motion trajectory diagrams for motion within the radial plane at each station, to permit a graphical identification of the surface waves present during different periods of the records. This required combining the two horizontal components of the acceleration records and computation of a radial horizontal acceleration record at seven stations where the recording sensors were not in the radial plane. The radial horizontal and vertical acceleration records at each station were then step integrated and combined to obtain radial particle motion trajectory diagrams. The particle motion

trajectory diagrams are provided here for only two sites. The particle motion trajectory diagrams for the remaining thirteen sites are given in AA (1980c). There are three factors important in interpreting the ground motions depicted by the particle motion trajectory diagrams. The first factor is the effect of the time step for each average displacement vector used to plot the particle motion trajectory diagrams on the resolution of higher frequency motion. Since the major objective in preparing the particle motion trajectory diagrams was to identify surface wave motion, the average vector was plotted for each 0.1 sec of the acceleration record. It follows that displacements of motions having a period of 0.5 sec (i.e., a frequency of 2 Hz) are defined by five points per cycle of motion along the plotted trajectory. However, displacements for motions having a period of 0.2 sec (i.e., a frequency of 5 Hz) are represented by only two points per cycle on the trajectory and will have poor resolution. In fact, depending on the time of arrival, motions having this period could be missed completely. Therefore, motions having a frequency of about 3 Hz will have poor resolution and motions having a frequency of 5 Hz and above will in general not appear on the particle motion trajectory diagrams.

Normally, the strong body wave motions have dominant frequencies between about 2 and 10 Hz for rock and firm soil sites at distances of 20 to 40 km from the epicenter. Therefore, for these sites, most of the body wave displacements will not appear in the particle motion trajectory diagrams. However, for deep soil sites, the dominant body wave frequencies may fall in the 1 to 3 Hz range and some of these displacements may appear. It should be remembered that the displacement half-amplitudes associated with 2 to 10 Hz motion are quite small. For a peak acceleration of 0.2 g, the half-amplitude displacement for 2, 5, and

10 Hz sinusoidal motion is 0.5 in., 0.08 in., and 0.02 in., respectively, but at 1 Hz the half-amplitude displacement is 2 in. It follows that much of the evaluation of body wave motion provided here has been inferred from the ground motion records and response spectra (see App. A, AA, 1980c) and not from the particle motion trajectory diagrams.

A second factor that should not be overlooked when evaluating the particle motion trajectory diagrams is that most of the stations considered in this study are several kilometers apart. Therefore, it was not possible to use average displacements for a group of stations to help minimize long period errors. The particle motion trajectories for each station must not, therefore, be considered to be a precise record of motion.

The third factor is the frequent assumption that all Rayleigh wave motion is retrograde elliptical at the ground surface. While this is theoretically true for the fundamental mode of Rayleigh wave motion, Drake and Mal (1972) have demonstrated that higher mode Rayleigh wave motions are frequently prograde elliptical motions. Therefore, both retrograde and prograde motions of a general elliptical shape and of a period of 1 or more seconds have been assumed to be some form of surface wave motion in the analysis that follows.

Two additional general comments apply to the particle motion trajectory diagrams given here. First, positive radial displacement has been taken as displacement away from the epicenter. Positive vertical displacement has been taken as displacement upward. At some stations this sign convention is opposite to that used by the California Institute of Technology in processing and plotting the acceleration records. However, it was decided that it would be less confusing if the same sign convention were maintained for all particle motion trajectory

diagrams rather than to provide agreement between the ground motion records and the particle motion trajectory diagram for each station. Second, the vertical displacement in each particle motion trajectory diagram is plotted as a function of the radial displacement with time as a parameter. The time in seconds is indicated at each second along the trajectory. Unfortunately, zero time was indicated by the Number 1 in these diagrams. Therefore, the time in seconds since the start of motion during the first 15 sec of motion is one less than the number indicated. All particle motion trajectory diagrams were divided into two or more diagrams for ease in reading, with the first division coming at the end of the first 15 sec of motion. The numbering indicating the time of motion starts over in the second diagram, with 1 indicating the displacement at the end of 15 sec of motion.

The procedure followed in making the qualitative estimate for each station follows. The overall results are summarized in Table 10-2 for eleven stations, which was within the scope of this study.

10.3.2 PACOIMA DAM

The Pacoima Dam strong motion recorder is located on a relatively narrow rock ridge that provides the left (south) abutment of a concrete arch dam. The instrument is housed in a corrugated metal vertical cylindrical structure 5 ft in diameter which was provided solely to protect the instrument. There is therefore negligible structural interaction effects on the records collected at this station. The instrument is mounted on a short concrete block cast on the surface of the rock abutment. Some concern has been expressed over the possible influence of the steep local topography and cracks that developed in the rock abutment on

the accelerations recorded during the 1971 San Fernando earthquake. The strongest peak horizontal ground acceleration (1.17 g) ever recorded for an earthquake was registered at this station during the 1971 San Fernando earthquake.

The Pacoima Dam is located about 9 km south of the instrumentally located epicenter of the 1971 San Fernando earthquake and is directly above the fault surface that dips to the north below the site at an angle of about 72 deg. The fault intersects the ground surface at the base of the mountain a few kilometers to the south of the station. The rock upon which the instrument is located is classified as a gneissic granite-diorite and probably has a shear wave velocity of about 5000 fps at the surface.

Particle motion trajectories in the vertical radial plane for the Pacoima Dam site are provided in Figure 10-3. Because of the topography and close proximity of the fault, the motions are complex, and individual types of wave forms are difficult to identify in the particle motion trajectory diagram. Perhaps the most direct approach to analyze the motions recorded at the Pacoima Dam site is first to identify the periods (or frequencies) that have significant amplification in the response spectra (AA, 1980c). Examination of the spectra for 2 to 10% of critical damping reveals significant amplification in both the horizontal and vertical response spectra at a period of about 5 sec. Since this period of motion is present in both spectral components and since it is a relatively long period, it has been assumed to represent the response of a surface (Rayleigh) wave.

Significant response is also noted at a period of about 2 sec on the vertical spectra, but the peak is at a period of about 1.5 sec in

TABLE 10-2. SUMMARY OF ESTIMATED CONTRIBUTION OF BODY AND SURFACE WAVE MOTION TO PEAK GROUND MOTIONS IN VERTICAL RADIAL PLANE

No	Station	Epicentral Distance, km	Component	No	Acceleration				
					Peak Accel g	Body Wave		Surface Wave	
						Contri, %	Freq, Hz	Contri, %	Period, Sec
1	Pacoma Dam	9	V H	1	0 71 1 17	95 95	3 2 5/10 *	5 5	5 5
2	8244 Orion Avenue	22	V H	2	0 17 0 25	65 65	3 1 5	35 35	6 2/4 5
3	15107 Vanowen Street	25	V H	3	0 11 0 12	95 25	3 2 5	5 75	5 2/5
4	15250 Ventura Blvd	29	V H	4	0 10 0 22	95 100	2 5 3	5 0	5 3
5	3838 Lankershim	31	V H	5	0 07 0 17	100 100	3 2 7	0 0	5 5
6	Griffith Park Observatory	34	V H	6	0 12 0 18	100 70	4/8 4	0 30	0 7/3 5 1
7	7080 Hollywood	35	V H	7	0 06 0 08	80 50	3 3 3	20 50	1 5/12 1 5/5
8	Hollywood Storage Blvd	36	V H	8	0 05 0 11	80 90	4 2	20 10	1 5 2 5
9	8639 Lincoln Ave	50	V H	9	0 04 0 03	85 -	3 5 -	15 -	2 -
10	5260 Century Blvd	52	V H	10	0 03 0 06	100 80	1 5/4 1 2/5	0 20	4 5 2 5
11	2516 Via Tejon, Palos Verdes	68	V H	11	0 02 0 04	100 100	3 1 3/3	0 0	1/4 1 5/4

No	Velocity					No	Displacement				
	Peak Veloc in /sec	Body Wave		Surface Wave			Peak Displa in	Body Wave		Surface Wave	
		Contri , %	Freq , Hz	Contri , %	Period, sec			Contri , %	Freq , Hz	Contri , %	Period, sec
1 V H	23 0 44 6	60 [†] 30 [†]	3 2 5	40 [†] 70 [†]	5 5/5	1	7 6 14 8	10 5	3 2 5	90 95	5 2/5
2 V H	12 6 11 8	0 50	3 1 5	100 50	2 5 2/4 5	2	5 8 5 9	0 10	3 1 5	100 90	3 2/4 5
3 V H	7 1 12 4	0 5	3 2 5	100 95	2 5 2/5	3	2 8 6 9	0 5	3 2 5	100 95	2 5 2/5
4 V H	3 7 11 1	55 10	2 5 3	45 90	5 3	4	1 7 5 3	5 5	2 5 3	95 95	3 3
5 V H	2 0 4 8	5 60	3 2 7	95 40	1 5/5 5	5	0 9 1 9	10 10	3 2 7	90 90	5 5
6 V H	2 9 8 0	0 0	4/8 2/4	100 100	0 7/3 5 1/4	6	1 3 2 9	0 0	4/8 2/4	100 100	0 7/3 5 1/4
7 V H	2 2 5 0	30 10	3 3 3	70 90	1 5/12 1 5/5	7	1 7 3 2	0 0	3 3 3	100 100	1 5/12 1 5/5
8 V H	2 4 6 7	50 50	1 5 2	50 50	1 5 2 5/4	8	1 5 3 4	10 10	1 5 2	90 90	4/12 2 5/4
9 V H	2 7 3 3	0 -	3 5 -	100 -	2/4 -	9	1 5 2 6	0 -	3 5 -	100 -	2/4 - [§]
10 V H	2 1 5 3	0 20	1 5/4 1 2	100 80	4 5 5	10	1 4 3 4	0 5	1 5/4 1 2	100 95	4 5 5
11 V H	0 9 1 5	0 0	3 3	100 100	1/4 4	11	0 6 1 0	0 0	3 3	100 100	1/4 4*

* Horizontal radial motions are inferred from oblique sensor and radial particle motion trajectory diagram

[†] Relative contribution of body and surface wave motion is questionable

[§] Horizontal radial motions are not available

(a) Particle motion during 0 to 15 seconds

(b) Particle motion during 15 to 30 seconds

FIGURE 10-3. PARTICLE MOTION TRAJECTORIES IN THE VERTICAL RADIAL PLANE FOR PACOIMA DAM — 1971 SAN FERNANDO EARTHQUAKE

the horizontal spectra.* Because of the length of the period and because the site is a rock site, this response ordinarily would be interpreted to be due to higher mode surface waves. The third peak of significant response occurs at about 3 Hz on the vertical spectra but occurs at 2.5 Hz on the horizontal spectra. Because of the relatively high frequency, these motions should be body waves with the dilatational (P) wave appearing in the vertical motions and the shear (S) wave appearing in the horizontal motions. Since the peak accelerations are high, the maximum half-amplitude displacements for these motions should be about 0.6 in. for the vertical motion and about 1.5 in. for the horizontal motion. Other peaks are noted at higher frequencies in both the vertical and horizontal response spectra for low damping. These are interpreted to be higher frequency body wave motions. Displacement motions with a 5 sec period are present in both the horizontal and vertical displacement time histories. These motions would be typical of surface wave motions.

A relatively simple Rayleigh wave motion appears to develop in Figure 10-3 during the first 3 sec (Points 1 to 4). Note that the motion starts out slowly with about 2.2 sec (Point 3.2) being required to reach the peak vertical downward displacement (the breaks in the trajectory occur at 0.1 sec intervals). At this point, the velocity increases with the peak horizontal displacement occurring at 2.5 sec (Point 3.5). After 2.5 sec, the horizontal

velocities increase rapidly (straight segments of trajectory are longer) until shortly after 3 sec (Point 4). These higher velocities are associated with the strong negative velocity pulses discussed above, which occur on both the horizontal and vertical velocity time histories between 2.5 and 3.5 sec.

Between 3.5 and 5.7 sec (Points 4.5 and 6.7 in Figure 10-3) two loops develop in the particle motion trajectory diagram that have the displacement characteristics of shear waves, except the period of the larger loop is abnormally long for body waves at a rock site, and both horizontal and vertical displacements are evident. These could be the result of higher mode surface waves, or could be due to a nonvertically propagating shear wave that could occur in close proximity to an inclined fault. It is concluded that it is the latter. The particle motion trajectory diagram indicates retrograde elliptical motion between 6 and 10 sec (Points 7 and 11) with higher frequency disturbances that result from body or higher mode surface wave motions.

At this point it is of interest to estimate the relative contribution of the body and surface wave motions to the peak accelerations, velocities, and displacements. The vertical motion time histories (AA, 1980c) show that the peak vertical ground acceleration and velocity appear to occur on the same cycle of motion. In the velocity time record, it can be observed that the higher frequency body wave velocity pulse is superimposed on a 5-sec surface wave pulse with both peaks occurring almost in phase. The peak vertical velocity for the 5-sec surface wave is estimated to be about 23 cm/sec (9 in./sec). The peak ground accelerations associated with this surface wave motion should be about 0.03 g, and the peak displacement should be about 7 in. (17 cm). The peak velocity associated with the body wave motion is

*The shift in period is attributed to the fact that the motion is elliptical and not circular. The particle therefore moves with a variable velocity during its cycle of motion. Also, the motion is transient and not steady state. It should be further noted that body and surface wave motions cannot normally be differentiated in the near field.

estimated to be about 35 cm/sec (14 in./sec). For 3 Hz motion, this velocity should be associated with a peak ground acceleration of about 0.65 g and a peak ground displacement of about 0.75 in. (2 cm).

Based on the above calculations, the peak acceleration, velocity and displacement, assuming the peaks are all in phase, would be 0.68 g, 58 cm/sec (23 in./sec), and 19 cm (7.5 in.). These should be compared with the peak recorded motions of 0.71 g, 58 cm/sec, and 19 cm that are indicated in the time motion histories given in AA (1980c). If the above assumptions are correct, the peak vertical ground acceleration resulted primarily from body wave motion with less than 5% being contributed by the surface wave motion. The peak vertical displacement resulted from surface wave motion with not more than 10% being contributed by body wave motion. However, the peak vertical velocity is estimated to have been strongly affected by both motions, with the body wave contributing 60% of the peak velocity and the surface wave contributing the remaining 40%.

The peak horizontal ground acceleration and velocity do not occur on the same cycle as occurred for the vertical motion but are about 5 sec apart. The peak horizontal velocity in this case was largely contributed by the strong pulse occurring between 2.5 and 3.5 sec, which is postulated to have been a surface wave motion with a period of about 1.5 sec. A higher frequency shear wave of about 2.5 Hz also appears on the velocity record, and these two motions are superimposed on a 5-sec period surface wave motion. It is not possible to make a close estimate of the relative contribution of each wave to the peak velocity, but it would appear that the two surface waves are of opposite sign and that the long period velocity is approximately equal to the body wave velocity. It is estimated that the body wave

contributes about 30% of the peak negative velocity. The total peak velocity would have been greater had the two body wave motions been in phase.

The peak horizontal acceleration and displacement do occur at about the same time at about 8 sec. It appears from the velocity record that the 5-sec period surface wave has a half amplitude of about 30 cm/sec (12 in./sec), and that the half amplitude of the 2.5 Hz body wave is also about 30 cm/sec (12 in./sec). Peak ground accelerations associated with these motions would be 0.05 g and 0.45 g, respectively, which totals only 0.5 g. Since 1.17 g was recorded, the remainder would have resulted from higher frequency body waves, which frequently occur on crystalline rock sites.

Based on the above assumptions, it is estimated that about 95% of the peak horizontal ground acceleration resulted from body wave motion with a high frequency spike contributing much of the peak. About 95% of the peak horizontal displacement is estimated to have resulted from surface wave motion, and about 70% of the peak horizontal velocity appears to have resulted from surface wave motion and the remainder from a body wave. However, there is some question concerning the composition of the major horizontal velocity pulse. The horizontal velocity estimate should therefore be considered questionable.

10.3.3 ORION AVENUE

The records considered in this study from 8244 Orion Avenue were recorded on the ground floor of the Holiday Inn, which is a seven-story reinforced concrete structure, 62 by 160 ft in plan dimension. The Orion site is located approximately in the center of the San Fernando valley where alluvial soils are estimated to be about 900 ft deep. The site is approximately

22 km south of the epicenter of the 1971 San Fernando earthquake and experienced strong surface wave motions during this earthquake. The particle motion trajectory diagram for this station is given in Figure 10-4. Ground motion records and response spectra are given in the appendix of AA (1980c).

Examination of the response spectra indicates that significant amplification of motion occurred at a period of 3 sec and at a frequency of 3 Hz for the vertical motion. The former should normally be associated with a surface wave and the latter with a body wave. On the horizontal response spectra, significant amplification occurs at periods of 4 to 5 sec and at 1.5 to 2 sec, and at a frequency of 1.5 Hz. The first two are probably associated with surface wave motion and the last (1.5 Hz) with body (shear) wave motion. It is of interest to compare the dominant shear wave frequency of 1.5 Hz for the soil site with the dominant shear wave frequency of 2.5 Hz identified with the Pacoima Dam rock site. This is apparently due to the difference in fundamental shear mode frequency of the two sites.

Examination of the vertical displacement time record indicates a predominant period of about 3 sec. It is of interest to note that the long period motions show conspicuously in the vertical acceleration record starting at about 16 sec. The long period motion appears in the velocity record, but the velocity record also shows that a secondary long period motion is providing secondary peaks in velocity and displacement at 8, 12, and 17 sec. Strong P-wave motions are present during the first 8 sec. The 3 Hz frequency of these motions can be most easily observed in the velocity record.

The particle motion trajectory diagram in Figure 10-4 provides important information on the surface wave motion. During the first

9 sec (Point 10), the motion is quite erratic, and the fact that the higher frequency motion is obscured may have contributed to the difficulty of interpreting the long period motion. The motion starts slowly and then moves quickly downward at the time strong P-waves are being recorded. The motion then reverses to become retrograde motion characteristic of a surface wave. However, by 6 sec a series of loops have developed that continue for 3 sec. These are probably a combination of higher mode surface waves and body wave motions. By 11 sec (Point 12) strong retrograde motion develops that continues for 15 sec. However, a strong shear wave motion appears to be superimposed on the surface wave motion at about 12 sec, which contributes the peak horizontal ground acceleration at about 12.5 sec (Point 13.5). This is judged to be a shear wave motion since it is accompanied with very little vertical motion.

It is of interest to note that the strong retrograde elliptical motion has its major axis nearly vertical rather than horizontal (between Points 5 and 10 in Fig. 10-4b). It is also noted that the surface wave motion is not steady state motion but varies in amplitude and frequency with time.

The period of the surface wave for the cycle that produced the peak vertical velocity and displacement is 2.5 sec. The peak surface wave acceleration that should accompany a 32-cm/sec velocity at a period of 2.5 sec is about 0.08 g (80 cm/sec/sec). Therefore, at least 80% of the peak ground acceleration accompanying the peak velocity and displacement is derived from the surface wave. The absolute peak vertical acceleration, however, occurs slightly before 4 sec at a time when the long period secondary peak displacement was occurring. The peak displacement was about 6 cm (2.5 in.). The assumption of a 6-sec period gives a peak surface wave acceleration

(a) Particle motion during 0 to 15 seconds

(b) Particle motion during 15 to 30 seconds

FIGURE 10-4. PARTICLE MOTION TRAJECTORIES IN THE VERTICAL RADIAL PLANE FOR 8244 ORION AVENUE — 1971 SAN FERNANDO EARTHQUAKE

of about 0.06 g (60 cm/sec/sec). The recorded peak was 167.5 cm/sec/sec (0.17 g). Therefore, it would appear that 35% of the peak vertical acceleration may have been contributed by the surface wave.

The peak horizontal motions were all obviously affected by the surface wave motion, but it is difficult to estimate the relative contribution. The half amplitude of the shear wave displacement indicated in Figure 10-4a at about 12.5 sec (Point 13.5) is about 0.85 in. This should be associated with a peak acceleration of about 0.17 g. Peaks of about 0.13 g are occurring both before and after this peak at the same frequency (1.5 Hz). Since 0.25 g was recorded, it is estimated that one-third (0.08 g) resulted from the surface wave and two-thirds from the body wave.

The peak horizontal body wave acceleration is estimated to about 0.13 g at the time the peak horizontal velocity and displacement occur. The peak velocity and displacement accompanying this acceleration at 1.5 Hz would be 5.5 in./sec (14 cm/sec) and 0.6 in. (1.5 cm), respectively. Peak values recorded were 30 cm/sec and 15 cm, respectively. At this time interval surface waves of at least two periods are present. Nothing is gained by attempting to determine the contribution from each surface wave. Therefore, it is estimated that about 50% of the peak horizontal velocity was contributed by the surface waves and at least 90% of the peak displacement was contributed by surface waves.

10.3.4 OTHER STUDIES

Descriptions of the remaining recording stations along the 4th azimuth (Fig. 10-1; also Table 10-1) are presented in brief form here. For each of these, the particle motion trajectory diagram is shown in AA (1980c), and the estimates of body and surface wave contribution to

the peak motion are provided in Table 10-2 above.

10.3.4.1 Vanowen Street

The strong motion recorder at 15107 Vanowen Street is located on the ground floor of the Presbyterian Hospital, which is a seven-story circular reinforced concrete building. This site is similar to that at Orion except it lies an additional 3 km from the epicenter of the 1971 San Fernando earthquake, the soil profile is estimated to be only 700 ft deep, and the basement rock and the Topanga formation are both rapidly decreasing in depth with increasing distance from the epicenter at this station. More than 15 cycles of strong Rayleigh wave motion were recorded at this station, with a period of about 3 sec. The particle motion trajectory diagram for this site is similar to that given in Figure 10-4 for the Orion station, except very strong secondary surface waves are superimposed on the longer period motion.

10.3.4.2 Ventura Boulevard

Three strong motion recorders are located at 15250 Ventura Boulevard, 14724 Ventura Boulevard, and 15910 Ventura Boulevard. The Ventura Boulevard sites are located at the southern edge of the San Fernando valley at a distance of 29 km from the epicenter of the 1971 San Fernando earthquake. In contrast to the sites at Orion and Vanowen, the unconsolidated alluvium at these sites is only about 65 ft in depth. The shallow alluvium terminates a short distance to the south of the Ventura Boulevard sites, as the uplifted basement complex forming the central core of the Santa Monica mountains gradually pinches off the alluvium and the underlying sedimentary rocks of the Modelo and Topanga formations. The inclined and tapered rock formations greatly reduce the amplitude of

the surface wave motion and give an inclination to the major axis of the particle motion trajectories, which indicates that the waves are radiating almost normal to the inclined rock formations. The estimates of the body and surface wave contribution to the peak motion are provided in Table 10-2 for the station at 15250 Ventura Boulevard; no estimates are provided for the stations at 14724 Ventura Boulevard and 15910 Ventura Boulevard, as the scope of this study did not permit this.

10.3.4.3 Lankershim Boulevard

The structure at this site is the 20-story reinforced concrete Sheraton Universal Hotel. Plan dimensions of the central tower are approximately 58 ft by 184 ft with the longitudinal axis of the building extending in the east-west direction. The lower three floors expand to plan dimensions of about 96 ft by 199 ft, but the central tower is separated seismically from the expanded lower three-story area (Blume, 1973a). An RFT-250 strong motion accelerograph recorded the ground motion considered in this study on the basement floor, which is a 5-in. concrete slab poured on grade approximately 16 ft below the normal ground surface. The recorder is centrally located in the building in the north-south direction but is located near the east end of the basement area. Spread footings which are continuous in the east-west direction under each row of columns provide the foundation. The footings are tied in the north-south direction with grade beams at each column line.

The site is located 31 km south of the epicenter of the 1971 San Fernando earthquake near the base of the northern slope of the Santa Monica Mountains; the site lies about 10 km east of the 15250 Ventura Boulevard site near the eastern end of the mountain range. The structure is founded upon the Topanga formation, which locally consists of a shallow

bed (10 to 20 ft) of soft sandstone underlain by a gray silty clay shale. The foundation is intersected by a fault zone which is approximately 20 ft in width and strikes to the north-east at an angle of about 70 deg. The Topanga formation dips steeply to the north in this region and is underlain by the basement complex which outcrops a short distance to the south.

10.3.4.4 Griffith Park Observatory

This station was not considered by Hanks but has been added to provide a station located on crystalline rock (similar to the Pacoima Dam record) on each side of the San Fernando Valley. The recorder at this station is located on the basement rock complex in the Moon Room of the Griffith Park Observatory, which is located near the summit of the Santa Monica Mountains. This site is 34 km from the epicenter of the San Fernando earthquake. The site is approximately 3 km from the soft sedimentary rock site at 3838 Lankershim and approximately the same distance from the alluvial soil site at 7080 Hollywood Boulevard. A shear wave velocity of about 4100 fps has been estimated for the basement rock at the Griffith Park Observatory (Duke et al., 1971).

10.3.4.5 7080 Hollywood Boulevard

At this site the strong motion recorder is located on the first floor of the Muir Medical Center office building, which is a 12-story reinforced concrete structure. Plan dimensions of the office tower are approximately 89 ft by 144 ft with the longitudinal axis extending in the north-south direction. Below the third floor level, the structure is expanded in plan in both directions to provide additional facilities (bank, pharmacy, restaurant, etc.). The lower floor is used as a parking basement but is above grade except on the north side of the

structure (Gates, 1973). The 1971 San Fernando earthquake was recorded at this station on an AR-240 accelerograph placed on the 4-in. concrete slab poured on the basement grade. The instrument is located approximately in the center of the building. The structure is supported on drilled and belled concrete caissons that extend about 26 ft below the basement floor level. Caissons are tied in two directions with 14 in. by 14 in. tie beams.

The site is located 35 km south of the epicenter of the 1971 San Fernando earthquake and is situated at the base of the southern slope of the Santa Monica Mountains. The upper 20 ft of the soil at the site is described as soft to moderately firm silty sands, composed of natural soils and fill material. The caissons for the foundation were therefore carried down about 26 ft to a firm silty clay. A boring indicated silty clays, sandy silts, and clayey sands to a depth of 75 ft, which is the depth of the boring (Gates, 1973). The site is near an inferred fault zone and the general area is covered with alluvium that has obviously washed down from the mountains. Terrace deposits are indicated to the south of the alluvial area, and to the south and east the Puente formation outcrops. The depth of the soils at the site are unknown but could be a few hundred feet (100 to 300).

10.3.4.6 Hollywood Storage Building (Basement)

Strong motion acceleration records were monitored both in the basement of the Hollywood Storage Building and in the parking lot approximately 100 ft west of the building during the 1971 San Fernando earthquake. The building is 14 stories high and was constructed prior to 1938. It is located on Highland Avenue near Santa Monica Boulevard. The building is 218 ft by 51 ft in plan with the longitudinal axis being oriented in the east-west direction. The total

height of the building exceeds 150 ft, which provides an aspect ratio of 3:1 in the north-south direction. The structure is founded on piles. The strong motion recorder, USCGS Standard, is mounted on the floor slab in the southwest corner of the basement. The basement floor slab is approximately 8 ft below the ground surface.

10.3.4.7 8639 Lincoln Boulevard

The RFT-250 strong motion recorder at this station is located in the basement of the 12-story reinforced concrete Airport Marina Hotel, which is located on Lincoln Boulevard approximately one block south of Manchester Avenue. The tower portion of this complex is approximately square in cross section and about 150 ft in plan dimensions. The first floor is essentially open to provide for vehicular traffic. This site is located approximately 50 km south of the epicenter of the 1971 San Fernando earthquake. Recent sand dune deposits cover the local area. Terrace deposits probably extend to a depth of several hundred feet at the site. A normal sequence of Upper and Lower Pliocene and Upper Miocene soft sedimentary rocks overlie the basement complex.

10.3.4.8 5260 Century Boulevard

The MO-2 strong motion recorder at this station is located on the first floor of the Airport Center building, which is a seven-story steel frame structure approximately 150 ft by 300 ft in plan with its longitudinal axis oriented east-west. This site is 52 km south of the epicenter of the 1971 San Fernando earthquake. The near surface soils are terrace deposits and probably extend to a depth of several hundred feet. Shear wave velocities of less than 1300 fps have been reported for soils near this site down to a depth of 70 ft (Duke et al., 1971). Sedimentary rock formations underlying

the terrace deposits are comparable to those indicated above for 8639 Lincoln Boulevard.

10.3.4.9 Via Tejon

The RFT-250 strong motion recorder at this station is located on the ground floor of a two-story brick masonry structure in Palos Verdes Estates. The site is 68 km south of the epicenter of the 1971 San Fernando earthquake. The area has very shallow Pleistocene alluvium overlying the Monterey shale, which is of Upper Miocene age. The structure probably rests on shale. The Monterey shale overlies the basement complex which is the Catalina schist of Mesozoic age which, due to local faulting, is probably within 4000 ft of the ground surface.

10.3.5 SUMMARY OF RESULTS

The relative contribution of the body and surface wave motions to the peak ground acceleration, velocity, and displacement recorded at the eleven stations considered along the south azimuth from the epicenter of the 1971 San Fernando earthquake are summarized in Table 10-2. The peak vertical and horizontal (radial) accelerations recorded at each station are listed in units of gravity, and the peak vertical and horizontal (radial) velocities and displacements computed by integrating the acceleration records are expressed in in./sec and in., respectively. The relative contribution to the peaks from each wave form is then indicated in percent. The dominant frequencies of the body wave contributions and the dominant periods of the surface wave contributions are also noted. The following observations have been drawn from the data in this table.

- a. Generally, 95 to 100% of the peak ground acceleration has resulted from body wave motion. Important exceptions have occurred at those sites exhibiting strong surface wave

motions (Orion, Vanowen, and Griffith Park) at distances of less than 30 km from the epicenter and at a site in close proximity to a faulted region where the geologic structure is complex (7080 Hollywood Boulevard). At these sites, it is estimated that 35 to 75% of the peak ground acceleration has been contributed by surface wave motion.

- b. Ninety percent or more of the peak vertical and horizontal (radial) displacements were contributed by surface wave motions at all stations.
- c. The contributions of body and surface wave motions to the peak velocities are quite variable, with the surface wave contributing 90% or more of the peak motion at only 55% of the stations. At some stations the surface wave contribution to the peak velocity was estimated to be 50% or less (Pacoima, Orion, Ventura, Lankershim, and Hollywood Storage Building).
- d. The body wave frequency contributing the peak horizontal ground acceleration appears to vary with site conditions. The two crystalline rock sites (Pacoima and Griffith Park) have dominant body wave frequencies of 2.5 and 4 Hz, respectively, but the peak horizontal ground acceleration at each site resulted from higher frequency motion (10 to 15 Hz at Pacoima; 8 Hz at Griffith Park). The deep soil sites had dominant frequencies between 1.5 and 2.5 Hz and the shallow soil sedimentary rock sites had dominant frequencies of 2.5 to 3 Hz.

e. The dominant surface wave periods for the various sites do not appear to be a function of near surface site conditions but are apparently determined by the deeper sedimentary rock profile. The most common fundamental mode period contributing to the

peak velocities and displacements varied from 4 to 5 sec. Higher mode surface waves having periods of 1.5 to 2.5 sec also made major contributions to peak ground velocities and displacements.

CHAPTER 11

EVALUATION OF GROUND MOTION CHARACTERISTICS

11.1 INTRODUCTION

Criteria to determine vibratory ground motions due to earthquakes are an important part of the seismic and geologic siting criteria document for nuclear power plants, Appendix A, 10 CFR, Part 100 (USAEC, 1973a). This document states that earthquake ground motion specifications at a site must be based on investigations of the seismicity and geology of the area, and local soil conditions. The consideration of local soil effects is clearly specified (Para. V-a-1):

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....

Methods for estimating vibratory ground motions are assessed here, along with a general procedure for specifying such ground motions for nuclear plant design purposes.

11.2 ESTIMATING VIBRATORY GROUND MOTIONS

Earthquake-induced motions are initiated by the release of energy in the earth's crust, which usually results from sudden slippage along an active fault. This energy takes the form of seismic wave emissions that propagate away from the earthquake source and through the geologic layers. The nature of these emissions near the source depends on the type of fault slippage and the extent, sequence, orientation, and location of the slipped area. As the waves travel through the surrounding soil and rock, they are further modified as they encounter layer interfaces, irregularities in the

surface topography, dissipative effects of the subsurface materials, and geometric spreading losses. Therefore, it is clear that many factors related to the source mechanism, source-site transmission path, and local soil conditions can affect the ground shaking at a site.

Because of the complexity of the problem, earthquake ground motions at a site cannot be predicted with certainty; instead, they must be estimated from the current library of strong-motion records and from site response analyses. The use of these techniques must be tempered by an awareness of numerous factors that influence ground shaking and of the limitations of current technology in accounting for these factors. Only in this way can a proper framework be established for reasonable engineering estimates of vibratory ground motions.

The earthquake ground motion criteria for a nuclear plant must establish peak accelerations, time histories, and spectrum shapes that represent the ground shaking that would be induced by the Safe Shutdown Earthquake (SSE) or Operating Basis Earthquake (OBE) for the site. These factors are established by site-independent or site-dependent procedures.

11.2.1 SITE-INDEPENDENT APPROACH

The site-independent approach develops vibratory motion criteria from past strong motion accelerograms recorded on sites that represent a variety of geologic characteristics and local soil conditions. Smoothed statistical composites of response spectra derived from these records form the basis of this method. These spectra typically must be scaled so that their zero-period acceleration corresponds to

the nuclear plant site's peak ground acceleration criterion, which, in turn, has been estimated from empirical relationships between acceleration and epicentral Modified Mercalli Intensity, or between acceleration, earthquake magnitude, and causative fault distance. Peak accelerations are most often used to measure the strength of the ground shaking in site-dependent as well as site-independent approaches (see Chapter 6). However, other parameters (such as peak velocity, spectrum intensity, etc.) may be more appropriate for this purpose (Housner, 1973).

The first site-independent spectrum shapes were devised by Housner (1959), and were followed by spectra developed by Newmark and Hall (1969). More comprehensive efforts along these lines by Newmark (1973) and Blume (1973b) were combined to provide a single set of recommended spectrum shapes (Newmark, Blume, and Kapur, 1973), which forms the basis for the AEC Regulatory Guide (RG) 1.60 used in many current nuclear plant applications (USAEC, 1973b). Comparisons of these spectra (5% damping) show that the Newmark/Hall and RG 1.60 spectra have similar shapes but are quite different from the Housner spectrum (Fig. 11-1). These dissimilarities can be traced

to differences in the numbers of records used to develop each spectrum shape and in the statistical bases from which these shapes were derived (see Table 11-1). For example, the Housner spectrum represents a mean (of two horizontal components of motion only) from 4 sets of records, while RG 1.60 is based on a mean plus one standard deviation statistical level from 25 sets of records. For RG 1.60, separate spectrum shapes were developed for horizontal and vertical motions. Table 11-1 also shows that these spectrum shapes were derived from records that represent a wide variety of local soil conditions.

11.2.2 SITE-DEPENDENT APPROACH BASED ON SITE-MATCHED RECORDS

Site-matched records correspond to an ensemble of accelerograms recorded at stations whose geologic and local soil characteristics more nearly match those of the nuclear plant site than do records used in the site-independent approach. Statistical composite spectra obtained from these site-matched records provide an additional basis for defining vibratory ground motion criteria.

Site-matched records should, ideally, be selected so that the various factors that affect ground shaking (i.e., source-mechanism, source-site transmission path, and local soil conditions) nearly match those of the nuclear plant site and its designated SSE and OBE events. In practice, however, the limited size of the current library of strong-motion records prevents this, and some of these site-matching requirements must invariably be relaxed. As a result, judgment becomes an important factor in defining those general site characteristics that might have the greatest effect on the ground shaking, and in selecting an ensemble of records that best represent these characteristics.

FIGURE 11-1. SITE-INDEPENDENT SPECTRA

TABLE 11-1. BASIS FOR DEVELOPMENT OF SITE-INDEPENDENT SPECTRA

Earthquake Data				Number of Components Considered in Spectrum Determination				Estimated Site Characteristics (Seed et al., 1974)	
Location of Epicenter	Date	Magnitude	Recording Station	Housner (1959)	Newmark/Hall (1969)	RG 1.60		Depth to Rock ft.	Site Classification
						Newmark (1973)	Blume (1973)		
Lower California	1934	6.5	El Centro	2	See Garcia and Roesset (1970)†		2	100	Stiff
Helena	1935	6.0	Helena				2	Rock	Rock
Imperial Valley	1940	6.3	El Centro	2		3	2	100	Stiff
Western 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			3		200	Stiff
			Storage Lot			3		200	Stiff
Eureka	1954	6.6	Eureka			3	2	500	Deep coh.
			Ferndale			3		500	Deep coh.
El Alamo, Baja	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	Temblor				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	Tokachi-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

†Garcia and Roesset 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).

*Estimated in SW/AA (1975).

11.2.3 SITE-DEPENDENT APPROACH USING SITE RESPONSE ANALYSIS PROCEDURES

A third approach for developing vibratory ground motion criteria is through analyses of the local soil response characteristics. Such analyses provide a means for estimating the effects that actual site soil conditions will have on the acceleration levels and the frequency

content of the ground shaking at any depth within the soil profile. The ability of the site response analyses to estimate depth-dependent site response is of particular value for soil/structure interaction analyses and for satisfying USNRC requirements for specifying input criteria at the base of the embedded nuclear plant structure. Such information cannot be directly obtained from the current library of strong-motion records.

Site response analyses involve three steps: definition of subsurface input motions, modeling of the site profile, and evaluation of the analysis results. These steps are described and assessed in Table 11-2. The table indicates that parametric analyses are necessary in order to examine the sensitivity of the site response to such uncertainties as those associated with specifying subsurface input motions and subsurface soil properties. Reasonable bounds on the dynamic response of the site can be established

if care and judgment are exercised in performing the required parametric analyses and in interpreting their results.

11 3 RECOMMENDED PROCEDURE FOR DEFINING VIBRATORY GROUND MOTION CRITERIA

Due to the complex nature of the earthquake-induced ground shaking, earthquake motions at a nuclear plant site can only be estimated. An important basis for providing

TABLE 11-2. SITE-RESPONSE ANALYSIS PROCEDURES

Step	Description	Assessment
1 Subsurface Input Motions	Input motions are based on an ensemble of rock-outcrop records or artificial rock motions whose acceleration levels are scaled using empirical data to represent the specified size of the SSE and OBE and the location of the site relative to the causative fault. These scaled records are used directly as input at the base of the model or, alternatively, are used to compute subsurface rock motions by deconvolution procedures (Reimer et al, 1973, Schnabel et al, 1972)	This is perhaps the most difficult step in the analysis procedure, due to a lack of subsurface records and to the sensitivity of rock-outcrop motions to such factors as surface topography, the directionality of the fault-slip process, rock properties, etc. Therefore, the ensemble of input motions should anticipate the full range of shaking that could occur at subsurface rock level.
2 Modeling of Site Profile	<p><u>Soil Material Behavior</u> Nonlinear properties of soil materials are often based on an equivalent linear representation. Equivalent moduli and viscous damping factors for each sublayer are adjusted through an iterative approach to be compatible with average strains induced in the sublayer by applied subsurface input motions (Seed, 1969). Other more elaborate material models, which defined soil properties as a function of stress, strain, and stress and strain history, have also been developed (Isenberg, 1972).</p> <p><u>Geometry of Site Profile</u> One-dimensional continuum or discrete-mass models are used to predict horizontal motions due to vertically propagating shear waves; such models are appropriate for use at horizontally layered sites. For sites where other types of waves are important, two-dimensional, finite-element procedures must be used.</p>	<p>Equivalent linear models are most widely used because of their simplicity and apparent ability, in previous studies, to provide reasonable results for a great variety of applications. The more elaborate models provide a more complete and theoretically correct representation of soil material behavior, however, their application is considerably more difficult and time consuming than equivalent linear models.</p> <p>When properly applied, one-dimensional models represent a useful engineering tool for analysis of horizontal motions at horizontally layered sites; at such sites, these models have yielded reasonable comparisons with measured records, and are relatively simple to apply. However, since these models only represent vertically propagating shear waves, their results will be deficient in the frequency ranges of other waves that may be present. In addition, they do not provide a direct means for computing vertical motions, which must be estimated from separate computations, judgment, or site-matched records. Two-dimensional models are more difficult to apply, and three-dimensional nonlinear dynamic models are still in the development stage.</p>
3 Assessment of Results	Analysis results can be provided at any depth in the site profile as response spectra and as time histories (or peak values) of motion, stress, and strain. Parametric studies should be carried out to establish the sensitivity of the site response to such uncertainties as those associated with specifying input motions or subsurface soil properties.	One of the main advantages of site response analyses is the variety of types of results that can be obtained.

such estimates is the current library of strong motion records; however, the local soil characteristics of the particular site being investigated, together with its design earthquake magnitudes and causative fault distances, may not be fully reflected by such records. Therefore, site response analyses can be an important aid in establishing vibratory ground motion criteria, but their results must be used with judgment because of inevitable difficulties that arise from estimating subsurface input motions and from establishing the properties of the site profile, particularly at substantial depths.

This means that data from strong-motion records and from site response analyses on an individual, site-by-site basis should be used for defining vibratory ground motion criteria. Because of the uncertainties associated with

each approach, the final selection of the criteria must be based on sound engineering judgment regarding the applicability of each data source to the particular site under investigation.

11.3.1 DESCRIPTION OF PROCEDURE

A recommended procedure (SW/AA, 1975) for defining vibratory ground motion criteria in accordance with requirements of Appendix A, 10 CFR, Part 100, has 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; and (6) comparisons with RG 1.60 and final selection of vibratory-motion criteria. This procedure is presented schematically in Figure 11-2 and summarized in Table 11-3. The

FIGURE 11-2. RECOMMENDED PROCEDURE FOR DEFINING VIBRATORY GROUND MOTION CRITERIA

TABLE 11-3 DESCRIPTION OF RECOMMENDED PROCEDURE

Step	Data Required	Information Derived	Procedures
1 Geologic and Seismologic Studies	Appendix A 10 CFR, Part 100 (USAEC, 1973a)	Definition of regional geology and tectonics, and the most significant earthquakes for site Identification of empirical data from existing earthquake records for estimating peak acceleration levels	Geologic Studies require the determination of the physiography, lithology, stratigraphy, structural geology, and geologic history of area, and the identification of capable faults and tectonic provinces Seismologic Studies include definition of the most significant earthquakes (dates of occurrence, magnitudes, intensities, correlations with capable faults or tectonic provinces, attenuation rates), and identification of empirical ground-motion data
2 Site Soils Investigations	SW/AA (1972)	Static and dynamic properties of the site soil and rock materials	Field Procedures consist of trenching, exploratory borings, and geophysical investigations to establish basic soil types, layer thicknesses, relative densities, body wave velocities, ground-water conditions, and depths to bedrock Laboratory Procedures test soil and rock samples to establish properties required for site response analyses, including strain-dependent shear moduli, damping ratios, etc
3 Site Response Sensitivity Studies	Data from Steps 1 and 2	Establishment of reasonable bounds on site response Identification of parameters with greatest influence on site response	A suitable model of the site is first developed, and parameters are identified whose effect on the site response may be significant (such as depth to rock, soil properties, or subsurface input motions) The effects of these parameters are then investigated by a series of site response analyses
4 Evaluation of Local Site Response Characteristics	Computed Results from Step 3	Definition of peak-acceleration criterion for site Development of MSD spectra representing site-response analysis results	The results obtained in Step 3 are evaluated Ensembles of site response spectra and peak accelerations (resulting from variations in parameters having greatest influence on site response) are identified These ensembles are analyzed statistically to obtain the MSD peak acceleration and response spectra The MSD peak acceleration is used with empirical data from existing records (Step 1) to define the peak acceleration criterion for the site The MSD spectra are used in Step 6 as a partial basis for defining site-criteria spectra
5 Selection of Site-Matched Records	Data from Steps 1, 2, and 4	Development of MSD spectra representing site-matched records	The current library of strong-motion records is examined to select an ensemble of records whose geologic, seismologic, and local soil conditions correspond closest to those identified for the site in Steps 1 and 2 Spectra from these records, when scaled to correspond to the peak-acceleration for the site (Step 4), are analyzed statistically to develop MSD spectra The spectra are used in Step 6 as a partial basis for defining site-criteria spectra
6 Comparisons with RG 1 60 and Final Selection of Vibratory Motion Criteria	RG 1 60 and results from Steps 4 and 5	Selection of final criteria spectra for site Generation of corresponding motion time histories (if required)	MSD spectra that represent site response analysis results (Step 4), site-matched records (Step 5), and RG 1 60 (scaled to peak-acceleration criterion of site) are compared Based on this comparison and on an assessment of the different sets of results as they apply to the site, final site-criteria spectra are developed Where required, corresponding time histories can also be developed from the site response analyses, if appropriate, or by generating artificial records that match the criteria spectra (Tsai, 1972)

detailed operations of each step can be adapted to technological advances due to additions to the library of strong motion records and improved site analysis procedures.

An important element of the procedure is the weighting of the spectral results from site response analyses, site-matched records, and RG 1.60 for the definition of the criteria spectra in Step 6. No set weighting procedure can apply to all possible siting conditions; instead, careful judgment must be incorporated on a site-by-site basis. Some of the judgment factors that influence the weighting process are (1) the degree to which seismologic, geologic, and local soil conditions at the site are represented in the current library of strong motion records, (2) an assessment of the ability of the site analytical model to transmit the kinds of seismic waves that influence ground shaking at the site, and (3) correlations of site soil conditions with the composite set of conditions represented by the RG 1.60 spectrum shape (see Table 11-1).

11.3.2 ILLUSTRATIVE EXAMPLE

An example application will illustrate the recommended procedure. A deep, cohesionless soil site has been selected for this example and, together with its design earthquake event, is analogous to conditions under which strong motion records were measured at 8244 Orion Boulevard (Los Angeles) during the 1971 San Fernando earthquake. For this example, 5%-damped horizontal-response spectra at the ground surface are assumed to comprise the vibratory motion criteria, although the procedure can be extended to provide more general criteria specifications. In addition, because of the analogous conditions at the example site and at Orion, this criteria spectrum will be compared to spectra from the Orion horizontal records.

Step 1: Geologic and Seismologic Studies.

The site is located in a seismically active area in southern California. Geologic and seismologic investigations indicate that the site SSE corresponds to a magnitude 6.5 event centered in a reverse-thrust fault zone located eight miles from the site. The corresponding Modified Mercalli intensity at the site is estimated at VII-VIII. Empirical data (Coulter et al., 1973; Housner, 1965) suggest peak accelerations in the range of 0.25 g to 0.30 g (Fig. 11-3). In addition, subsurface input motions for use in site response analyses will be estimated in Step 3 using deconvolution procedures; therefore, it is necessary to estimate peak accelerations on rock outcropping also located eight miles from the center of a magnitude 6.5 earthquake. For these conditions, empirical data (Schnabel and Seed, 1972) suggest a peak rock acceleration of 0.36 g (Fig. 11-3a).

Step 2: Site-Soils Investigations.

The soil profile for the site consists of deep deposits of dense and gravelly sands that extend to sandstone and shale rock at a depth of about 800 ft (Fig. 11-4). These deposits exhibit shear wave velocities that increase with depth to a maximum of about 1500 to 2000 ft/s at the soil/rock interface. The rock beneath the soil deposits has a shear wave velocity of about 2800 ft/s (SW/AA, 1975). Layering characteristics in the vicinity of the site are reasonably horizontal and uniform. Strain-dependent shear moduli and damping factors for these materials can be represented by the Seed/Idriss curves provided in SW/AA (1972).*

The primary uncertainties in the site soil properties are judged to be related to the shear wave velocity of the dense-to-gravelly sands

* At an actual nuclear plant site, such strain-dependent properties would be established from field and laboratory test programs.

(a) Acceleration attenuation curves for magnitude 6.5 earthquake

(b) Acceleration vs. Intensity data (Coulter et al., 1973)

FIGURE 11-3. EMPIRICAL DATA FOR ESTIMATING PEAK ACCELERATIONS

FIGURE 11-4. SOIL PROFILE FOR EXAMPLE SITE

extending from 250 ft to 800 ft below the ground surface. To account for these uncertainties, two sets of soil properties will be considered in site-sensitivity studies (Step 3). These are termed "lower-bound properties" and "upper-bound properties" and are denoted by the curves marked as "dense sand" and "gravelly sand" in the figure. All properties for the other site soil and rock materials are assumed to remain unchanged.

Step 3: Site-Response Sensitivity Studies.

Since the site contains essentially horizontal layering (Step 2), a one-dimensional model based on vertically propagating shear waves can be used to analyze the horizontal response of the site. In addition, an equivalent linear material model, based on the strain-dependent properties developed in Step 2, was used. In the analytical procedure, an ensemble of rock-outcrop records was identified and scaled to correspond to the site SSE. Deconvolution procedures (Schnabel et al., 1972) were applied to these scaled rock records to compute corresponding motions at the depth of the subsurface soil/rock interface. For these surface motions as input to the Orion site model, ground-surface responses were computed (AA, 1975b).

The primary variables investigated in these sensitivity studies were the subsurface input motions and the soil properties in the dense-to-gravelly sand layer. These variables were investigated by subjecting the upper-bound and

lower-bound soil conditions (Step 2) to subsurface input motions derived from horizontal records measured at Pacoima Dam and Castaic during the 1971 San Fernando earthquake and scaled to a peak acceleration of 0.36 g (Step 1). The peak horizontal accelerations and response spectra (see AA, 1975b) computed at the site ground surface from this matrix of cases are given in Table 11-4.

TABLE 11-4. PEAK ACCELERATIONS FROM SITE RESPONSE SENSITIVITY STUDIES

Soil Properties	Rock Outcrop Input Motions		Peak Ground Acceleration, g's
	Record	Component	
Upper Bound	Pacoima Dam	S 16 E S 74 W	0.240 0.269
	Castaic	N 21 E N 69 W	0.284 0.283
Lower Bound	Pacoima Dam	S 16 E S 74 W	0.196 0.249
	Castaic	N 21 E N 69 W	0.250 0.243

Notes: 1. Rock outcrop motions scaled to peak accelerations of 0.36 g (Step 1)

2. Composite results from ensemble of computations:

Mean = 0.252 g
MSD = 0.280 g

Step 4: Evaluation of Local Site Response Characteristics. The sensitivity studies carried out in Step 3 indicate that the peak ground accelerations are generally insensitive to the assumed variations in the input motions and soil properties. Ground surface response spectra are also insensitive to the assumed changes in the soil properties, but are more strongly influenced by the character of the input motions. This influence is greatest at frequencies below 2 Hz, although in the 2-to-10 Hz frequency range of particular importance for nuclear plants, significant effects can also be noted. Furthermore, when compared to the spectra from the rock outcrop motions, the computed ground response spectra exhibit consistently larger amplitudes at low frequencies (below 1 to

2 Hz) and reduced amplitudes at high frequencies (above 2 to 4 Hz); this trend is qualitatively consistent with past comparisons of deep soil and rock-outcrop response characteristics, as observed from analyses and statistical composites of measured records (Schnabel et al., 1972; Seed et al., 1974).

Step 4 must define a peak-acceleration criterion for the site and a mean plus one standard deviation (MSD) spectrum from the results of the site response analyses. An MSD peak acceleration of 0.28 g was computed from the ensemble of analysis results in Table 11-4 and was compared to the acceleration estimates from empirical data obtained in Step 1 (0.25 g to 0.30 g). Although acceleration levels were similar for both sets of data, the analysis results provide the more reliable estimates; therefore, a peak acceleration of 0.28 g was selected as the site criterion. The corresponding MSD spectrum from the analysis results, scaled to a zero-period acceleration of 0.28 g, will be used in Step 6 as a partial basis for developing the criteria spectrum for the site.

Step 5: Selection of Site-matched Records.

No adequate ensemble of records in the current library of strong motion records sufficiently represents all of the seismologic, geologic, and local soil conditions at this example site. Therefore, the selection of site-matched records was based primarily on local soil conditions, in which spectra from records measured at deep, cohesionless soil sites were used. The MSD spectrum shape developed by Seed et al. (1974) from an ensemble of 30 records obtained at deep, cohesionless soil sites was selected for application at the example site. This spectrum shape was scaled so that its zero-period acceleration corresponded to the 0.28 g peak-acceleration criterion developed for the site in Step 4.

Step 6: Comparisons with RG 1.60 and Final Selection of Vibratory Motion Criteria. The final development of the vibratory motion criteria for the example site is based on comparisons of MSD spectra from the site response analysis and the site-matched records with the RG 1.60 spectrum shape. This comparison, shown in Figure 11-5a, indicates that (1) the site-dependent spectra from the analysis results and the site-matched records compare reasonably well (within about 25%) over the entire

frequency range of interest; and (2) the site-dependent spectra exceed RG 1.60 at frequencies below 3 Hz and fall slightly below RG 1.60 at frequencies above 3 to 6 Hz.

Causes for the differences between the site-dependent spectra and RG 1.60 can be attributed to the nature of the site conditions for the records used in the development of RG 1.60. Approximately one-third of these records were obtained at deep, cohesionless soil sites (see Table 11-1); the remaining records were measured on either rock or stiff soil sites whose depths to rock are considerably less than that at the example site. Therefore, the RG 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 11-5a. Consequently, although differences between the RG 1.60 and site-dependent spectra are not substantial in the primary frequency range for nuclear plants (2 to 10 Hz), the site-dependent results are judged to be more representative of the ground response for this particular site. For this reason, the site criteria spectrum is selected to be a smoothed best fit that envelopes the site-dependent spectra given in Figure 11-5a. This smoothed spectra is shown in Figure 11-5b, and differs only slightly from RG 1.60.

Results from this site evaluation are compared to spectra from the Orion records of horizontal motion in Figure 11-6. This comparison indicates that the mean spectra from both sets of site-dependent results compare closely to the Orion spectra. The criteria spectrum is shown to be conservative when compared to the Orion spectra, but not unreasonably so.

(a) Comparisons of site-dependent and RG 1.60 spectra

(b) Comparison of criteria spectrum with RG 1.60

FIGURE 11-5. CRITERIA SPECTRUM FOR EXAMPLE SITE

FIGURE 11-6. COMPARISONS OF SITE-DEPENDENT RESULTS WITH ORION SPECTRA

CHAPTER 12

DESIGN EARTHQUAKE MOTIONS

Earthquake requirements for the design of nuclear power plants and test facilities are set forth in the Standard RDT-F9-2, which is issued by the Reactor Research and Technology Division of the Department of Energy. This document lists applicable codes, standards, classification requirements for design, seismic load applications, and analytical and design requirements. This standard requires that nuclear plant structures, systems, and components be designed to accommodate seismic loadings resulting from a Safe Shutdown Earthquake (SSE) and an Operating Basis Earthquake (OBE), which are defined in Code of Federal Regulations 10 CFR 100, Appendix A. Recommended procedures (AA, 1980a) are provided here to define design vibratory ground motions for these design earthquakes (see also Chapters 8 and 9 above).

12.1 INTRODUCTION

Procedures for defining design earthquake ground motion have evolved over nearly a half century, dating from the deployment of the first earthquake strong motion recording instruments following the Santa Barbara, California, earthquake of 1925. Since strong earthquakes occur infrequently, the collection of reliable instrumentation data has been slow and sporadic. As a result, early ground motion prediction procedures were based largely on judgmental postulations and very limited instrument data. These procedures have been invalidated by information obtained from later earthquakes, but even today there are serious limitations in strong motion instrumentation data. These limitations are somewhat accounted for by the use of probabilistic techniques.

The requirement for a probabilistic assessment of the earthquake vibratory ground motion hazard associated with a proposed nuclear plant site is not defined in the federal regulations but is inferred by the high standards of safety and reliability required for containment and safe shutdown during and following a major seismic event. Lack of a clear understanding of this requirement results from the fact that the procedures set forth in the Code of Federal Regulations, 10 CFR 100, Appendix A, for the selection of design earthquakes for nuclear plant sites are completely deterministic. As a result, extremely conservative design earthquakes are frequently selected, and the level of risk incorporated into the design earthquake ground motions is normally not determined until the design has been nearly completed and the level of reliability achieved by the overall design is estimated. One result is that nuclear plants are currently being designed to different levels of seismic risk.

The annual probability of experiencing the mean peak ground motions (acceleration and velocity) used to scale design response spectra should be determined at the time the design earthquake ground motions are established. In this manner, overly conservative, as well as unconservative, design ground motion criteria can be detected and corrected at an early stage. The probabilistic assessment requires five steps:

1. Review of the geology and seismic history of the region.
2. Prediction of maximum earthquake magnitudes possible for each source or source area.

3. Development or selection of earthquake vibratory ground motion attenuation relationships with distance from the earthquake source.
4. Development of relationships (probabilistic) between earthquake recurrence rates and earthquake magnitudes for the contributing sources or source areas.
5. Statistical calculations to establish a relationship between the yearly probability of exceedance and the intensity level (i.e., peak horizontal ground acceleration and velocity or Modified Mercalli Intensity) of vibratory ground motion possible at the site.

The probabilistic assessment differs from the deterministic assessment in requiring the final two steps.

12.2 GUIDELINE RECOMMENDATIONS

Design earthquake motions are usually defined through the use of design response spectra. Recommended design response spectra can be established in a probabilistic manner following the five-step assessment procedure just outlined. These steps will be expanded upon here, although the first two steps are only briefly discussed because they are the work of the geologist and the seismologist, respectively. Once earthquake ground motion attenuation relationships (Step 3) are established, a probabilistic assessment of vibratory ground motion (Step 4) can be made, and from these, design response spectra calculated (Step 5).

12.2.1 GEOLOGICAL AND SEISMOLOGICAL INVESTIGATION

A thorough and complete geological and seismological investigation of the site and region

is required in order to identify capable faults and source areas that contribute to the seismic hazard associated with a proposed site. Seismic and geologic siting criteria are set forth in 10 CFR 100, Appendix A.

12.2.2 PREDICTION OF MAXIMUM EARTHQUAKE MAGNITUDES

The maximum earthquake magnitudes (or epicentral Modified Mercalli Intensities) must be established for each capable fault or source area for the proposed site. Seismic siting criteria are set forth in 10 CFR 100, Appendix A. When earthquake magnitudes are established deterministically, the results are generally an assignment of overly conservative magnitudes. In order to select earthquake magnitudes that are rational and believable, the maximum magnitudes (or epicentral Modified Mercalli Intensities) for each contributing source or source area should be based upon the following:

- a. The seismic history of the relevant fault zones, source areas, and host tectonic province
- b. Geologic and geomorphic evidence of recent fault movements, e.g., offset strata, scarps, scarp-like features
- c. Comparison of the seismic history of the zones and province with similar structural and tectonic environments elsewhere
- d. Empirical fault length-fault displacement-magnitude relationships (e.g., Bonilla and Buchanan, 1970)

An additional recommendation is that the maximum magnitudes (or epicentral Modified Mercalli Intensities) selected should be plotted on the recurrence curves (see Step 5) to establish that the maximum values are rational and believable events.

12.2.3 EARTHQUAKE GROUND MOTION ATTENUATION RELATIONSHIPS

Relationships are presented here that express the rate at which the strength of the vibratory ground motions attenuate with distance from the epicenter, or center of energy release, of earthquakes. These relationships are used for deterministic as well as for probabilistic prediction of design ground motion. Ground motion attenuation relationships are normally expressed in terms of mean peak ground acceleration, velocity, and displacement, as a function of earthquake magnitude and source distance. The effect of site subsurface conditions on ground motion strength parameters is also a consideration. For regions where strong ground motion acceleration records are not available, the distance attenuation relationships are generally expressed in terms of Modified Mercalli Intensities.

The following subsections provide recommended guidelines (AA, 1980a) for ground motion attenuation relationships according to commonly used classifications.

12.2.3.1 Peak Horizontal Ground Acceleration

Peak horizontal ground acceleration/distance attenuation relationships have been presented by Bernreuter (1977), Donovan (1973b), Housner (1965), McGuire (1978), Schnabel and Seed (1973), Trifunac (1976), and Werner, Ts'ao, and Rothman (1979); see also Chapter 8 above. The following guidelines are recommended:

- a. Based on the scatter in the available data, the limited number of records available for rock sites and the uncertainties in the classification of site subsurface conditions at recording sites, site subsurface conditions have not been demonstrated to be a significant correlation factor in

regression analyses of peak ground acceleration as a function of magnitude and source distance. Therefore, for rock and stiff soil sites it is recommended that peak ground acceleration be assumed to be independent of site subsurface conditions. Adjustments in peak ground acceleration for soft clay and deep cohesionless soil sites may be made if justified by special studies.

- b. The (adjusted) Werner relationship (Eq. 12-1) with $E = 1$ is recommended as a guideline for regions of reverse faulting in western United States where focal depths of 10 to 15 km are anticipated, accompanied with surface manifestations of faulting. This relationship should be assumed to be equally applicable (at the ground surface) to rock and to stiff soil sites.

$$\ln a' = 12.38 - \frac{32.85}{M} - 1.01 \ln R + 1.72 E - 0.38 (\ln R) E \quad (12-1)$$

where

- $\ln a'$ = Natural logarithm of the peak horizontal ground acceleration, in./sec²
 M = Richter magnitude
 R = Hypocentral distance, km
 E = A constant based on fault conditions

- c. In regions of normal and strike-slip faulting in western United States, lower peak ground accelerations than indicated by Equation 12-1 with $E = 1$ may be justified. However, it is recommended that values not be used that are less than given by Equation 12-1 with $E = 0.545$ unless supported by studies based on regional

acceleration records from historical earthquakes.

- d. In regions where earthquakes have significantly different focal depths than typical of California earthquakes, and in regions where Modified Mercalli Intensity attenuation relationships are significantly different than experienced in California, the direct applicability of peak horizontal ground acceleration/magnitude/distance relationships based essentially on California earthquakes is questionable. In these regions, Modified Mercalli Intensity attenuation procedures are recommended (see Sec. 12.2.3.6).

12.2.3.2 Peak Horizontal Ground Velocity

Peak horizontal ground velocity/distance attenuation relationships have been presented by McGuire (1978), Trifunac (1976), Seed et al. (1976), and Werner, Ts'ao, and Rothman (1979). The following guidelines are recommended:

- a. The (adjusted) Werner relationship (Eq. 12-2) with $E = 1$ is recommended as a guideline for regions of reverse faulting in western United States where focal depths of 10 to 15 km are anticipated, accompanied with surface manifestations of faulting. The effect of site subsurface conditions should be considered in peak ground velocity predictions. All predictions should be based on a careful study of the local geology and site surface conditions that determine the susceptibility of the site to surface wave motions.

$$\begin{aligned} \ln v' &= 8.60 - 31.99/M \\ &- 0.66 \ln R + 2.98E \\ &- 0.66 (\ln R) E \\ &+ 0.07 (\ln R) S_D \\ &- 0.15 (\ln R) S_R \end{aligned} \quad (12-2)$$

Terms not defined previously in Equation 12-1 are:

$\ln v'$ = Natural logarithm of the peak horizontal ground velocity, in./sec

S_D = Deep soil site classification factor (equals 1 for deep soil sites and 0 for all other sites)

S_R = Rock site classification factor (equals 1 for rock sites and 0 for all other sites)

- b. In regions of normal and strike-slip faulting in western United States where strong surface waves are not anticipated, lower peak velocities than given by Equation 12-2 with $E = 1$ may be justified. However, it is recommended that values not be used that are less than given by Equation 12-2 with $E = 0.545$. When strong surface waves are anticipated, it is recommended that $E = 1$ be assumed in all cases.
- c. In regions where earthquakes have significantly different focal depths than typical of California earthquakes and in regions where Modified Mercalli Intensity attenuation relationships are significantly different than experienced in California, direct applicability of peak horizontal ground velocity/magnitude/distance relationships based on California earthquake data is questionable. In these regions, Modified Mercalli Intensity attenuation procedures (see Sec. 12.2.3.6) are recommended.

12.2.3.3 Peak Horizontal Ground Displacement

Peak horizontal ground displacement attenuation relationships have been presented by McGuire (1978), Trifunac (1976), and Werner, Ts'ao, and Rothman (1979). The same guidelines are recommended for peak ground displacement as given above for peak ground velocity except that Equation 12-3 should be used in place of Equation 12-2.

$$\begin{aligned}\ln d' = & 5.89 - 26.87/M - 0.36 \ln R \\ & + 3.11 E - 0.68 (\ln R) E \\ & + 0.06 (\ln R) S_D - 0.17 (\ln R) S_R\end{aligned}\quad (12-3)$$

All terms in this equation are as previously defined, except $\ln d'$ is the natural logarithm of the peak horizontal ground displacement in inches.

12.2.3.4 Peak Vertical Ground Motions

The mean peak vertical ground acceleration, velocity, and displacements can be assumed to be two-thirds of the respective mean peak horizontal components of motion, or the correlation equations given above can be used with the following adjustments in the constant of each equation:

In Equation 12-1, 11.71 should be substituted for 12.38

In Equation 12-2, 7.85 should be substituted for 8.60

In Equation 12-3, 5.30 should be substituted for 5.89

12.2.3.5 Mean Plus One Standard Deviation Ground Motions

Mean attenuation relationships for peak horizontal ground acceleration, velocity, and displacement are recommended for predicting site ground motions, because current response spectra amplification factors are based on mean peak ground motions. However, mean plus one standard deviation ($M + \sigma$) peak ground motions

are frequently required for other purposes. It is recommended that Equations 12-1, 12-2, and 12-3 be used as guidelines to obtain ($M + \sigma$) ground motions with the following adjustments in the constant of each equation:

In Equation 12-1, 12.95 should be substituted for 12.38

In Equation 12-2, 9.13 should be substituted for 8.60

In Equation 12-3, 6.50 should be substituted for 5.89

12.2.3.6 Modified Mercalli Intensity Attenuation Relationships

In regions where there are no strong motion acceleration records available, which is typical of the eastern two-thirds of the United States, isoseismal maps of historical regional earthquakes are used to establish the rate of attenuation of earthquake ground motion with distance. In these regions, it is convenient to express historical and design earthquakes in terms of epicentral Modified Mercalli Intensities (MMI). Regional MMI/distance attenuation relationships are developed from regional isoseismal maps based on the average distances from the epicenter to the isoseismal lines. Since there are many factors other than strength of ground motion that affect the assigned MMI values of historical earthquakes, some adjustment in MMI may be required to better reflect the rate of attenuation of strength of motion (see Chapter 7 above).

The relationships for mean peak horizontal ground acceleration given in Figure 12-1 are recommended for isoseismal MMI values of VIII and below. Recommendations for MMI/mean peak horizontal ground velocity and ground displacement are provided in Figures 12-2 and 12-3, respectively. These curves are based on Trifunac and Brady (1975) data and the mean peak ground motion/distance relationships given above. These guideline relationships should be

FIGURE 12-1. GUIDELINE ISOSEISMAL MMI/MEAN PEAK HORIZONTAL GROUND ACCELERATION RELATIONSHIPS

FIGURE 12-2. GUIDELINE ISOSEISMAL MMI/MEAN PEAK GROUND VELOCITY RELATIONSHIPS

FIGURE 12-3. GUIDELINE ISOSEISMAL MMI/MEAN PEAK GROUND DISPLACEMENT RELATIONSHIPS

assumed to be representative of average conditions in California (i.e., comparable to the relationships evaluated for $E = 0.545$). When the regional MMI/distance attenuation relationships are significantly different than California MMI/distance relationships, some adjustment in the MMI/peak ground motion guideline relationships may be required to better represent the regional attenuation characteristics. These adjustments must be based on special studies for each region.

12.2.3.7 Summary

In many regions of eastern United States, present deterministic procedures of 10 CFR 100, Appendix A require that nuclear facilities be designed for epicentral ground motions, and

MMI/distance attenuation relationships are not required. When the design earthquake can be specified in terms of magnitude, the mean peak ground motions resulting from Equations 12-1, 12-2, and 12-3 for $E = 0.545$, with R set equal to the estimated focal depth, are recommended as guidelines for regions where there is no evidence of surface faulting. When the design earthquake must be specified in terms of epicentral MMI values, the MMI/mean peak ground motion guideline relationships given in Figures 12-1, 12-2, and 12-3 can be used as guidelines if the anticipated focal depths are 20 km or more and there are no surface manifestations of faulting. The peak vertical motions shall be taken as two-thirds of the peak horizontal motions as a guideline in

regions where MMI data must be used as a basis for ground motion predictions.

The $(M + \sigma)$ peak ground motions are ordinarily not used to scale response spectra but may be of value in meeting other design requirements. Standard deviation values have been given by Trifunac and Brady (1975) for peak ground acceleration, velocity, and displacement, but these values have been computed for each individual MMI area. The standard deviation for an MMI area should be greater than for an isoseismal MMI since the former has a distance variation effect (i.e., variation in peak motion with distance across the MMI area). Additional study is needed to evaluate this effect quantitatively. For the present, it is recommended that the logarithms of the mean values for the peak ground acceleration, velocity, and displacement found from Figures 12-1, 12-2, and 12-3, respectively, be increased by the standard deviation values given in Table 12-1 (from Werner, Ts'ao, and Rothman, 1979) to obtain the logarithm of design $(M + \sigma)$ values.

TABLE 12-1. STANDARD DEVIATION VALUES

Ground Motion	Standard Deviation
Acceleration	0.57
Velocity	0.53
Displacement	0.61

12.2.4 PROBABILISTIC ASSESSMENT

The probabilistic assessment requires two additional steps: the development of a stable recurrence relationship for earthquakes as a function of magnitude or epicentral MMI for each source or source area; and the performance of probability calculations.

The most important requirement in the development of the recurrence relationship for earthquakes for a site assessment is

careful attention to the historical record of earthquakes. However, even though historical accounts of large earthquakes may extend over a period of nearly 200 years, the population density has not been sufficient until the last 50 to 100 years to guarantee that all intermediate magnitude earthquakes have been noted. Also, adequate instrumentation to detect and record low magnitude earthquakes in most regions of California has been in existence for less than 50 years and does not exist in most regions of the United States today. Extreme care must therefore be exercised in interpreting the historical data.

Mathematical models can perform the probability calculations, which should be based on mean peak ground motion attenuation curves to be consistent with the amplification factors used to scale $(M + \sigma)$ design response spectra, which are also based on mean peak ground motions (AA, 1980a). Advances in modeling techniques mean that relationships between magnitude and fault length, and fault location and focal depth can all be considered. In general, the models are sophisticated and are limited only by present knowledge of earthquake phenomenology.

12.2.5 DESIGN RESPONSE SPECTRA

To establish the design ground motions in a deterministic manner, the mean peak ground acceleration and velocity are predicted for the site based on the design earthquake (Steps 1 and 2) and the distance attenuation relationships (Step 3). These values are then used to scale design response spectra. However, a preferred treatment is that the probability of the site experiencing these mean peak values of ground motion first be determined so that overly conservative or unconservative values can be detected and corrected at an early stage. At present, acceptable probability

values for the design earthquake ground motions are not available. However, values having an annual probability of occurrence of 1×10^{-4} are recommended as a guideline for SSE when the calculations are based on mean peak ground motion distance attenuation curves.

Procedures for developing design response spectra have been presented by Housner (1959); Newmark and Hall (1969, 1976); Regulatory Guide 1.60 (USAEC, 1973b); Mohraz (1976); Seed, Ugas, and Lysmer (1976); Guzman and Jennings (1976); and Werner, Ts'ao, and Rothman (1979). At present, Regulatory Guide 1.60 response spectra for horizontal response are recommended guidelines for site independent response spectra. The Newmark and Hall (1976) procedures when based on an acceleration:velocity:displacement ratio consistent with the Standard Earthquake ratio of 1.0 g:48 in./sec:36 in., give comparable site independent horizontal response spectra.

The most important improvement in procedures for developing design response spectra at this time is to use the site dependent mean peak ground motions given in Section 12.2.3 as scaling factors rather than the Standard Earthquake ratio of peak values. For deterministic design criteria, it is recommended that the amplification factors given in Table 12-2 be applied to site independent design values of peak horizontal ground acceleration and velocity obtained by the procedures given in Section 12.2.3 to obtain guideline horizontal response spectra. The procedure is illustrated in Figure 12-4. A significant reduction in response will usually result for rock and intermediate (or firm soil) sites below frequencies of about 2 Hz from that given by Regulatory Guide 1.60 or Newmark and Hall (1976) procedures that are based on Standard Earthquake peak ground motion ratios. Higher response than

obtained by these site-independent procedures will frequently be obtained below frequencies of about 2 Hz for deep soil sites when site-dependent ground motions are used. The same procedures are recommended as guidelines for the construction of vertical response spectra. The vertical ground motions may be taken as two-thirds of the horizontal ground motions computed as indicated in Section 12.2.3.4.

Response spectra can be constructed using peak ground accelerations and velocities computed from a probabilistic assessment (see Sec. 12.2.4). When response spectra are based on mean peak ground motions that have been selected probabilistically to represent values that have an annual probability of 1×10^{-4} of occurrence, the resulting spectra should be corrected for frequencies above about 20 Hz to the $(M + \sigma)$ peak ground acceleration in order to maintain a uniform level of conservatism in response at all frequencies (see Fig. 12-4). The difference between the mean and $(M + \sigma)$ peak horizontal ground acceleration is comparable to the difference between the peak and "effective peak" ground acceleration that has confused communication between geological engineers and seismologists on one hand, and the geotechnical and structural design engineers on the other (see Page et al., 1972). Geological engineers and seismologists are well aware that peak ground accelerations at a site may be significantly greater than the peak ground accelerations frequently used by the geotechnical and structural design engineer to scale

TABLE 12-2. AMPLIFICATION FACTORS
(from Newmark and Hall, 1976)

Percent of Critical Damping	Spectrum Amplification Factors	
	Acceleration	Velocity
0.5	5.10	3.84
2.0	3.66	2.92
5.0	2.71	2.30
7.0	2.36	2.08
10.0	1.99	1.84

FIGURE 12-4. COMPARISON OF SITE-DEPENDENT GUIDELINE RESPONSE SPECTRA WITH NEWMARK AND HALL (1976) RESPONSE SPECTRUM

design response spectra. The reason that lower (mean) values are correctly used by the latter, however, is that the amplification factors used to scale $(M + \sigma)$ response spectra are based on the lower (mean) values. However, current procedures for scaling design response spectra yield response above 20 Hz that is less conservative than the response below 20 Hz unless the above correction is made.

When a probabilistic assessment is used to establish the peak design ground motions, the results are normally presented in terms of the annual, or service life, probability of occurrence of the mean peak ground acceleration (and velocity), or in terms of the annual, or service life, probability of occurrence of the isoseismal Modified Mercalli Intensity. Figure 12-5 gives an example for a California site in which mean peak ground acceleration is expressed in terms of annual probability, and

Figure 12-6 gives an example in which the service life probability of occurrence of the isoseismal Modified Mercalli Intensity is provided for a site in the southern Appalachian seismic zone. It is hoped that in the future, acceptable annual or service life probabilities will be specified by regulatory agencies for design earthquakes. When this is done, the appropriate design acceleration and velocity, or isoseismal Modified Mercalli Intensity, can be selected, based on the required probability level. Only in this way can nuclear plants all be designed to the same level of seismic risk. Until this has been done, the results of the probabilistic assessment can only be used to check the probability level of the values estimated for design by present deterministic procedures.

Values having an annual probability of not being exceeded of 99.99% are recommended as

FIGURE 12-5. PROBABILITY OF ANNUAL OCCURRENCE OF MEAN PEAK GROUND ACCELERATION AT HYPOTHETICAL SITE IN CALIFORNIA

FIGURE 12-6. MODIFIED MERCALLI INTENSITY PROBABILITY RELATIONSHIPS FOR HYPOTHETICAL SITE IN THE SOUTHERN APPALACHIAN SEISMIC ZONE, FIRM SITE CONDITIONS

reasonable values for the Safe Shutdown Earthquake (AA, 1980a). This is equivalent to a value that has a 50-year service life probability of 99.5% of not being exceeded and to an annual probability of occurrence of 1×10^{-4} . Using this probability level, the Safe Shutdown Earthquake (SSE) for the California site could be based on a peak horizontal ground acceleration of 0.5 g (see Fig. 12-5) and the SSE for the southern Appalachian seismic zone site could be based on an isoseismal Modified Mercalli Intensity VII + 1/2 (see Fig. 12-6).

For the California site considered above, the mean peak ground acceleration (and velocity) having an annual probability of occurrence of 1×10^{-4} has been used to scale the design response spectra given in Figure 12-7. The

spectrum has been adjusted for frequencies above about 20 Hz to the $(M + \sigma)$ peak ground acceleration that is a value of 0.9 g. It is of interest to note that nuclear power plants in the coastal region of California are currently based on Regulatory Guide 1.60 spectra scaled to peak ground accelerations of 0.67 to 0.75 g.

For the site in the southern Appalachian seismic zone considered above, the isoseismal MMI 'VII + 1/2 can be used to select the mean peak ground acceleration and velocity needed to scale the design response spectra. Figures 12-1 and 12-2 indicate a mean peak ground acceleration and velocity for rock or firm soil sites of 0.15 g and 6.5 in./sec, respectively. Assuming these values would be applicable to the site, the spectrum given in Figure 12-8 for

FIGURE 12-7. $(M + \sigma)$ RESPONSE SPECTRUM ON 1×10^{-4} ANNUAL PROBABILITY MEAN PEAK GROUND MOTIONS — HYPOTHETICAL SITE IN CALIFORNIA

FIGURE 12-8. $(M + \sigma)$ RESPONSE SPECTRUM SCALED FROM 1×10^{-4} ANNUAL PROBABILITY MEAN PEAK GROUND MOTIONS — HYPOTHETICAL SITE IN SOUTHERN APPALACHIAN SEISMIC ZONE

intermediate site conditions would apply, except for frequencies above 20 Hz. For frequencies above about 20 Hz, the response spectrum has been raised to the $(M + \sigma)$ peak ground acceleration. This spectrum is compared in Figure 12-8 with a Regulatory Guide 1.60 spectrum scaled to 0.25 g, which represents the level of response currently being provided for several nuclear plants in this region.

It should be apparent that when deterministic peak ground motion values are used to scale design response spectra that are higher than obtained from a probabilistic analysis

based on mean ground motion attenuation curves, an overly conservative response will be obtained, except for frequencies above about 20 Hz. It is noted that currently only the deterministic procedure is an acceptable regulatory procedure.

Recent work in near-field cases included a statistical analysis of 741 actual earthquake records and resulted in recommendations for computing the response spectra for near-field cases (AA, 1981). See Chapter 8 above, Table 8-1.

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