Proceedings of
SEMINAR ON NEW DEVELOPMENTS IN
EARTHQUAKE GROUND MOTION
ESTIMATION AND IMPLICATIONS FOR
ENGINEERING DESIGN PRACTICE
Los Angeles • San Francisco • Seattle • New York • Memphis

APPLIED TECHNOLOGY COUNCIL

Funded by
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Proceedings of
ATC-35 Seminar on New Developments in
Earthquake Ground Motion Estimation and
Implications for Engineering Design Practice

Los Angeles, California
January 26, 1994

San Francisco, California
January 27, 1994

Seattle, Washington
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New York, New York
February 9, 1994

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February 10, 1994

by
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Preface

In January and February 1994, the Applied Technology Council (ATC) conducted a series of five regional seminars on "New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice." The seminar series served as the initial activity in a larger U. S. Geological Survey-sponsored project to "Transfer U. S. Geological Survey Research Results into Engineering Design Practice" (ATC-35 Project).

The five initial seminars, designed for practicing structural and geotechnical engineers, were conducted in Los Angeles, California (January 26, 1994), San Francisco, California (January 27, 1994), Seattle, Washington (February 2, 1994), New York, New York (February 9, 1994), and Memphis, Tennessee (February 10, 1994). The purpose of each seminar was to provide comprehensive, but practical region-specific information on earthquake potential and the characteristics of expected ground shaking, with a special emphasis on issues relevant to the determination and mapping of design ground motions.

This report contains the technical papers presented at the initial five seminars. Specific paper topics included:

- Regional earthquake risk (focused on the region in which the seminar was conducted);
- Strong ground motion estimation (new techniques for estimating ground motions as a function of earthquake source, travel path, and site parameters, with emphasis on problems specific to the particular region); and
- Implications of new knowledge and new developments for engineering practice (specifically applicable to geotechnical engineering and structural engineering—the design of buildings and bridges).

The complete program for each seminar location, which contains paper titles, authors and panelists, is provided in Appendix A.

Applied Technology Council gratefully acknowledges the many individuals who contributed to the success of the seminar series. Maurice Power, Project Director and Co-Principal Investigator, Charles C. Thiel, Co-Principal Investigator, and Chris D. Poland, Structural Engineering Consultant, developed the seminar program and identified paper authors. Steering Committee members Arthur D. Frankel, Thomas H. Heaton, Thomas L. Holzer (USGS Project Officer), I. M. Idriss, Klaus H. Jacob, William B. Joyner, Helmut Krawinkler, Bijan Mohraz (ATC Board Representative), Allan R. Porush, Paul G. Somerville, Randall G. Updike, and Nabih Youssef provided overall guidance and direction. The affiliations of these individuals are provided in Appendix B.

Applied Technology Council also gratefully acknowledges the ATC staff for their assistance in planning and conducting the seminar. Patty Christofferson, Manager of Administration and Public Relations, selected the seminar meeting sites and organized the publicity effort. Staff members Karen Johnson and Bernadette Mosby distributed announcements and registered participants.

Christopher Rojahn
ATC Executive Director
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REVIEW OF POTENTIAL EARTHQUAKE SOURCES IN SOUTHERN CALIFORNIA

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Abstract

The tectonics of southern California are dominated by the relative motion of the Pacific Ocean and North American plates producing one of the highest levels of seismicity in the contiguous United States. The motion between the plates is translational, dominantly on the San Andreas fault system. A 160 km left step in the San Andreas fault, the "Big Bend", complicates the tectonic structure and gives rise to diffuse seismicity across the whole region. The seismogenic faults can be grouped into strike-slip faults parallel to the San Andreas that accommodate the plate boundary motion and reverse and thrust faults that accommodate the compression caused by the step. The Los Angeles basin lies at the intersection of the two regimes and is affected by earthquakes from both types of faults. The largest earthquakes (about magnitude 8) occur on the San Andreas fault but many other faults, both strike-slip and reverse, may be capable of infrequent, large (magnitude 7.5 or even larger) earthquakes. The rate of seismicity in the last 60 years suggests that the repeat time for magnitude 7 events somewhere in southern California is about 28 years.

Introduction

Southern California straddles the boundary between the Pacific Ocean and North American plates. The relative motion between the plates is primarily horizontal at a rate of about 48 mm/yr (Minster and Jordan, 1978; DeMets and others, 1987) two-thirds of which occurs on the San Andreas fault. A 160-km left step in the San Andreas fault spreads the deformation over a large area, including numerous normal, strike-slip and reverse faults. On a broad scale, the North American-Pacific plate boundary in California is a transform fault that extends from the Gulf of California to Cape Mendocino (Figure 1). The San Andreas fault and the transform plate boundary end at the Mendocino Triple-Junction in northernmost California. North of Cape Mendocino, the spreading center and subduction zone of the Juan de Fuca plate lie between the North American and Pacific plates. Another spreading center lies south of southern California in the Gulf of California, creating parts of the Pacific and Rivera plates. The transform faults of that spreading system merge into the San Andreas fault system near the Imperial Valley and the Salton Sea (Figure 1).

The relative motion of the plates in southern California is primarily accommodated on the right-lateral strike-slip faults of the San Andreas fault system (Figure 2), that is comprised of the San Andreas fault itself and a number of subparallel faults including those offshore in the Continental Borderland (Weldon and Humphreys, 1986). The San Andreas fault has the highest slip rate in this system, with slip of 25 mm/yr to 35 mm/yr (e.g., Sieh and Jahns, 1984; Weldon and Sieh, 1985). South of, and truncated by the Transverse Ranges, the San Jacinto, Elsinore, Newport-Inglewood faults and several probable offshore faults, parallel the San Andreas to the west. Northeast of the Transverse Ranges numerous smaller strike-slip faults strike northwest through...
the Mojave desert east of the San Andreas, transmitting part of the relative plate motion from the spreading centers south of the Imperial Valley and the southernmost San Andreas fault northeast to the Basin and Range province (e.g., Dokka and Travis, 1990b).

Comparing the trends of the San Andreas fault in the Coachella Valley and the Carrizo Plain (Figure 2) shows the offset of 160 km in the plate boundary in southern California. Accommodation of this step is reflected in many of the structures of southern California (e.g., Davis, 1983). The San Andreas fault itself changes strike, striking west-northwest through the step. Many of the subparallel faults lie on strike of the offset part of the fault — the San Jacinto and Elsinore faults along strike from the Carrizo plain and the faults of the Mojave desert along strike from the Coachella Valley. The compression across the step is taken up in large part by the reverse and thrust faults of the Transverse Ranges and the Los Angeles basin. These faults strike primarily east-west or east-northeast, sometimes with a component of left-lateral slip and slip rates in the submillimeter to several millimeter range.

In 61 years (1932 through 1992) of southern California earthquakes, 70 sequences have been recorded that included at least one earthquake of $M_L \geq 5.0$ (Hutton and Jones, 1993), giving an average rate of 1.1 $M_L \geq 5.0$ earthquake sequences per year. No significant variations in this rate have been documented over that time (Hutton and Jones, 1993). The average recurrence of $M \geq 6$ events is 5 years and the average recurrence of $M \geq 7$ events is 28 years. Several groups have estimated the overall earthquake hazard in southern California from geologic and geodetic data. The Working Group on California Earthquake Probabilities (1988) estimated the probability of large earthquakes on the San Andreas fault system. In southern California, this includes about half of the large ($M \geq 6.5$) events but few of the moderate, damaging ($5 < M < 6.5$) events. Incorporating what was known about the approximately 300 other faults capable of producing damaging earthquakes, Wesnousky (1986) mapped the probability of shaking exceeding 10%g across California. Ward (1994) modeled the seismic hazard by matching moment release rates determined from geodetic measurements. The Southern California Earthquake Center is integrating these approaches and preparing a summary of the earthquake hazard in southern California (Jackson and others, 1993). All of these approaches predict an overall rate, similar to that determined from the seismic record.

The most often reported magnitude for an earthquake in southern California is the local magnitude ($M_L$), determined from the photographic Wood-Anderson seismograms (Richter, 1958; Hutton and Boore, 1987; Hutton and Jones, 1993). In the case of earthquakes of magnitude 6.0-6.5 this magnitude scale may saturate. Seismic moment is a measure of the size of an earthquake based on the area of fault rupture, the average amount of slip, and the shear modulus of the rocks offset by faulting. Seismic moment is usually determined as part of the inversion of waveforms for the parameters of the earthquake source (Aki and Richards, 1980). An empirical relationship has been found between magnitude and seismic moment ($M_0$) (Hanks and Kanamori, 1979) and from this relationship, moment magnitude ($M_w$) has been defined. Following Hutton and Jones (1993), the magnitudes in this paper are local magnitudes below $M_6$ and moment magnitudes above $M_6$.

As has long been observed (e.g., Allen and others, 1965), the spatial pattern of microseismicity differs from the distribution of damaging earthquakes in southern California (Figure 3). Some Quaternary faults capable of major earthquakes such as the San Jacinto fault, also have a high level of microseismicity, but other major faults like the San Andreas fault are quiet at the microseismic level during much of the interseismic period. In spite of the recent improvements in location capability, a map of well located earthquakes in southern California "still looks like a shotgun has been fired at it" (Allen, 1981; Figure 3). All types of focal mechanisms have been recorded in southern California but northwest-striking right-lateral strike-slip and east-west reverse faults are the most common (Figure 4).
The complicated tectonic structure of southern California leads to a diversity of potential earthquake sources. In this paper, we address four major tectonic divisions of southern California -- the San Andreas fault system, the Transverse Ranges, the Mojave desert, and the Sierra Nevada and southern Basin and Range. We summarize the work of numerous authors on the seismotectonic structure and consider possible future earthquakes in each region.

San Andreas Fault System

San Andreas Fault. The San Andreas fault extends for 550 km in southern California (Table 1) and was responsible for the largest earthquake of the region, the 1857 M7.8 Fort Tejon earthquake (Figure 2, Table 2). In northern and central California, the San Andreas is a clearly delineated feature, striking northwest approximately parallel to the direction of plate motion, N35° W (e.g., Hill and others, 1991; Figure 1). Near Fort Tejon, along the northern edge of the Transverse Ranges, the fault changes strike to N70° W, in the so-called “Big Bend”. The main trace of the fault is clearly followed from there east-southeast to the northern end of the San Gorgonio Pass region where the fault splits into several strands, with the Banning and San Bernardino Mountain strands as the two most recently active features (Matti and others, 1985; 1992). Southeast of San Gorgonio, the multiple strands rejoin to form the Coachella Valley segment, and the strike of the fault resumes its northwest trend.

The rupture zone of the M7.8 1857 Fort Tejon earthquake on the San Andreas fault extended from Parkfield in central California, through the “Big Bend” near Fort Tejon, south to Cajon Pass (Sieh, 1978). The coseismic slip during the 1857 earthquake varied along strike, from 9 m near Wallace Creek, and 6 m around Fort Tejon, to 3-4 m near Palmdale (Sieh, 1978). No large earthquake (M ≥ 7.0) has been documented in the historic record (since 1749) for the San Andreas south of San Bernardino (the San Bernardino and Coachella Valley segments) (Allen, 1968).

The occurrence of moderate earthquakes and the lack of a clear surface trace in the San Gorgonio Pass region led some researchers to suggest that this part of the fault is incapable of large earthquakes (Allen, 1968; Wallace, 1970). However, field studies have shown that the Coachella Valley segment of the fault, southeast of San Gorgonio Pass, does produce large to great earthquakes (Sieh, 1986). Moreover, the hypocentral distribution of the 1986 North Palm Springs earthquake showed that, in spite of the complicated surface geology, the rupture surface of the Banning strand at depth continues on the N60°W trend that is the local trend of the San Andreas fault farther southeast (Jones and others, 1986). The maximum depth of seismicity is offset along a line connecting the San Bernardino and Banning strands (Jones and others, 1993). This strengthens the hypothesis of Matti and others (1985) that the San Bernardino and the Coachella Valley segment of the Banning fault, which both strike N60° W, may join at depth as a continuous fault system.

All reaches of the San Andreas fault south of Parkfield are capable of large or great earthquakes. Sieh and others (1989) suggested that segments of the San Andreas rupture separately or with adjoining segments at different times, leading to a range of possible magnitudes in each earthquake. The best estimates for San Andreas earthquakes are from magnitude about 6.8 to 8.0, depending on how many segments rupture together in an event. Recurrence intervals have been established from paleoseismic data at 5 sites so far on the southern San Andreas fault (Sieh and others, 1989; Fumal and others, 1993; Grant and Sieh, 1994; Sieh, 1986). The slip rate varies little, 25-35 mm/yr, but the average slip per event is more variable. Average recurrence intervals at the different sites on the San Andreas fault are range from 140 to 210 yr, although actual intervals between pairs of events have been as short as 45 years and as long as more than 300 years. Because of its high slip rate and great length, the San Andreas is without question the fastest repeating fault with the largest possible events and thus the greatest risk to southern California.
The San Andreas fault appears to be seismically quiescent between large earthquakes (Figure 3). The few small earthquakes recorded near the fault often exhibit oblique reverse or normal mechanisms and appear to occur on secondary fault structures (Jones, 1988).

Imperial Valley Fault. The Imperial Valley is one of the most seismically active regions of southern California with several damaging earthquakes before 1932. Two large earthquakes have occurred on the Imperial fault since 1932 (Table 2), a M6.9 event in 1940 (Neumann, 1942) and a M6.5 earthquake in 1979 (e.g., Johnson and Hutton, 1982). The epicenter of the 1940 earthquake was at the northern end of the Imperial fault, near the juncture with the Brawley Seismic Zone (Trifunac and Brune, 1970), and rupture was to the south, with the largest displacements occurring just north of the international border (Reilinger, 1984). In contrast, the 1979 earthquake nucleated south of the border, near the juncture of the Imperial fault with the Cerro Prieto Seismic Zone, and ruptured largely to the north (e.g., Archuleta, 1982; Hartzell and Heaton, 1983; Langbein and others, 1983; Sharp and others, 1982). In general, the 1979 earthquake re-ruptured the northern part of the 1940 earthquake fault rupture. Anderson and Bodin (1987) suggest that repeated magnitude 6 to 7 earthquakes, along with as much as 5 mm/yr of creep (Louie and others, 1985), on the Imperial fault account for a large percentage of the motion between the Pacific and North American plates at that latitude.

Sediments preclude a direct measurement of the slip rate of the Imperial fault but geodetic results suggest a rate as high as 30 mm/yr (WGCEP, 1988; Table 1). The small inter-event time between the 1940 and 1979 earthquakes, and apparently high slip rate on the Imperial fault suggest that its earthquake potential is high. The Working Group on California Earthquake Probabilities assigned a 30-year probability of 50% to this fault (WGCEP, 1988). The Imperial fault exhibits a moderate level of microseismicity, with almost all events showing right-lateral northwest-striking mechanisms (Figure 3).

Brawley Seismic Zone. Moderate earthquakes of magnitude 5 to 6 are common in the Brawley Seismic Zone that connects the Imperial Valley and San Andreas faults (Figure 2 and 3) (e.g., Johnson and Hill, 1982). Focal mechanisms of most swarm earthquakes in the Imperial Valley are northwest right-lateral strike slip (Figure 4) (Hutton and Johnson, 1981) even though the overall zone strikes north. Imperial Valley swarms typically have complex spatial-temporal patterns; the Westmorland sequence of 1981 (Hutton and Johnson, 1981) involved activity on at least seven distinct fault planes. The faults in the Brawley Seismic Zone are thought to be short enough that earthquakes much larger than 6-6.5 are unlikely.

San Jacinto Fault. The San Jacinto fault zone is a major tectonic feature both structurally and seismically. It is a member of the San Andreas fault system and strikes northwest for more than 200 km, parallel to the plate boundary, with the second highest slip rate in southern California at 12 mm/yr (Rockwell and others, 1990; Table 1). It merges with the San Andreas fault to the northwest at Cajon Pass and with the Imperial fault to the southeast. Its surface trace is complex with many anastomosing and splaying strands. It is predominately a right-lateral strike-slip fault, but some segments display significant amounts of dip-slip offset (Sharp, 1967).

The San Jacinto fault zone is seismically the most active structure in southern California at all magnitude levels less than 7.0. It has produced at least nine events of M_l ≥ 6.0 since 1890; four of these since 1932 (1937, 6.0; 1942, 6.6; 1954, 6.4; 1968, 6.5) (Figure 2, Table 2). The fault zone, therefore, has an average recurrence interval of about 10 years for magnitude 6.0 and larger events, with the longest interval between consecutive events being 19 years. The last large events on the San Jacinto fault system have been the Borrego Mountain earthquake (M6.5) on April 9, 1968 (e.g., Allen and Nordquist, 1972) and the Superstition Hills earthquake (M6.6) on November 24, 1987 (e.g., Magistrale and others, 1989). The San Jacinto fault will continue to be a source of frequent M6-7 earthquakes. Even larger earthquakes are thought unlikely because of
the discontinuous nature of the surface trace of the fault. WGCEP (1988) determined the probability of a moderate earthquake somewhere on the fault in 30 years to be 50%.

The San Jacinto fault has the highest level of microseismicity of any fault in southern California (Figure 3). More detailed seismicity maps and cross-sections reveal that the earthquakes occupy a diffuse band about the fault trace and do not sharply define individual fault planes. Background activity is generally concentrated in clusters at points of complication in the fault zone, such as fault segment junctions and terminations. Focal mechanisms (Figure 4) indicate slip planes consistent with the strike of the fault zone, even though much of the microseismicity clearly occurs on secondary structures.

Elsinore Fault. The Elsinore fault is the third most active member of the San Andreas system with a slip rate of 5mm/yr (Rockwell, 1989; Table 1). The May 1910 Temescal Valley earthquake (M6) is the only significant, historic earthquake associated with the Elsinore fault. The most likely segmentation of the fault would probably lead to earthquakes of about magnitude 7. The Elsinore fault is characterized by a moderate level of background seismicity although many of the small events near the fault have focal mechanisms suggesting they are occurring on secondary structures (Figure 4). The high slip rates and moderate event sizes lead to relatively short recurrence intervals of a few hundred years (Table 1).

Newport-Inglewood Fault. The Newport-Inglewood fault, located in the western part of the Los Angeles basin, is well studied because several major oil fields are adjacent to it (Wright, 1991). It was the source of the 1933 Long Beach earthquake (M6.4) that produced right-lateral slip of about 50 cm along 25 km (Hauksson and Gross, 1991). The fault can be divided into two segments, the south segment that broke in 1933 and the north segment that extends from Signal Hill to the Santa Monica fault. Both segments are 25-30 km long and may break in earthquakes of magnitude 6-6.5. The possibility of both segments breaking in one large earthquake is small but cannot be excluded. Guptill and Heath (1981) stated that the strike-slip or horizontal component of motion along the fault was 0.5 mm/yr since late Miocene and early Pliocene. To the south of Newport Beach, the Newport-Inglewood fault projects seaward into a system of faults having similar trends that can be traced more than 250 km south into Baja California. This part of the Newport-Inglewood fault could cause seismic shaking in the Los Angeles area as well as along the coastal area extending south to San Diego. These submillimeter slip rates mean that the probability of larger earthquakes are much lower than the previous 3 faults or several of the major thrust faults of the Transverse Ranges. Small earthquakes are common near the Newport-Inglewood fault with more strike-slip mechanisms to the south and more reverse mechanisms to the north (Hauksson, 1987).

Palos Verdes Fault. The Palos Verdes fault coincides with the Torrance-Wilmington fold and thrust belt along most of its length and is interpreted to be either a right-lateral strike-slip fault or a steeply dipping reverse fault with oblique displacement (e.g., Ziony and Yerkes, 1985; Davis and others, 1989). Although numerous efforts have been made to find horizontal surficial offsets on the Palos Verdes fault, no clear evidence for horizontal offsets has been found (Darrow and Fisher, 1983). Small earthquakes with strike-slip focal mechanisms have occurred near the Palos Verdes fault as it heads offshore, both in San Pedro Bay and Santa Monica Bay (Hauksson, 1990). Vertical offsets documented from uplifted marine terraces show a vertical uplift rate of 0.1-0.8 mm/yr (Patterson and Freeman, 1990). Because almost no evidence for strike-slip motion has been found and the vertical motion appears to be small, the Palos Verdes fault thus appears to accommodate only a small fraction of the total convergence being taken up by the southwest flank of the basin. Alternatively, moderate-sized or large earthquakes on the Palos Verdes fault may rarely rupture up to the surface.

Faults of the Continental Borderland. Several strike-slip faults parallel to the San Andreas system have been recognized offshore from southern California (e.g., Legg, 1985). The
largest recent earthquake on one of these faults was the 1981 Santa Barbara Island earthquake (M5.5) (Corbett, 1984). Because they are offshore, the data are limited, but these faults are likely capable of moderate to large events. However, the best estimates of slip rates are that they are probably comparable to the Newport-Inglewood and thus much lower than many other faults in southern California.

Transverse Ranges

The central and western Transverse Ranges trend east-west from the San Andreas fault in the east to Point Conception in the west. Three major systems of seismically active, west to west-northwest striking reverse faults have been recognized in the Transverse Ranges (Figure 2). The middle system are a group of Quaternary oblique reverse faults that dip steeply to moderately northward. This system of faults includes the Santa Cruz Island, the Anacapa Dume, the Santa Monica, the Raymond Hill and the Cucamonga faults (Ziony and Yerkes, 1985). A second system of Quaternary reverse faults branches north from the Cucamonga fault and extends to the west-northwest along the Sierra Madre, San Fernando and Santa Susana faults. This system can be traced across the Ventura basin and offshore from Santa Barbara to merge with the Hosgri fault (Bird and Rosenstock, 1984). The 1987 Whittier Narrows earthquake (M5.9) drew attention to another system of blind thrust faults buried beneath the sediments of the Los Angeles basin, south of the Raymond Hill and Cucamonga faults (Hauksson and Jones, 1989). The surface expression of these systems are a series of anticlinal hills, but slip at depth occurs on faults. This fault system assumes major importance in studies of earthquake hazard because it passes directly under downtown Los Angeles.

The 1971 San Fernando earthquake is the only historic earthquake in the Los Angeles basin that is known to have caused surface rupture. The focal mechanism showed reverse faulting with a component of left-lateral strike slip motion (Whitcomb., 1973). The mainshock caused surface faulting along the 19 km long San Fernando member of the Sierra Madre fault zone (e.g., Heaton, 1982). The 1991 Sierra Madre earthquake was also caused by thrust faulting along the Sierra Madre fault zone (Hauksson, 1994). The 1987 Whittier Narrows earthquake occurred on a previously unrecognized, concealed fault with pure thrust motion on a gently north-dipping plane (Hauksson and Jones, 1989). The 1988 and 1990 Upland earthquakes were caused by left-lateral slip on the San Jose fault that splays southwest from the main frontal fault of the central Transverse Ranges (Hauksson and Jones, 1991). These left-lateral faults allow small fragments from the Peninsular Ranges block to move westward around the Transverse Ranges.

Many faults in the Transverse Ranges pose a particular threat to the Los Angeles basin that lies at the transition from the strike-slip faults of the continental borderland (such as the Newport-Inglewood and Palos Verdes faults) to the reverse and thrust faults of the Transverse Ranges (Figure 5). The major faults are:

Sierra Madre Fault Zone. The 110 km-long Sierra Madre fault zone follows the base of the San Gabriel mountains from Cajon Pass to San Fernando in a series of seven arcuate, 13-20 km-long fault segments (Crook and others, 1987). Aftershocks from the 1971 San Fernando and 1991 Sierra Madre earthquakes show that the downdip width of these segments is about 23-28 km (Hauksson, 1994). The repeat time for earthquakes has been estimated along the San Fernando segment to be 200 years (Bonilla, 1973). Similarly, along the Cucamonga segment the repeat time was estimated to be 700 years by Matti and others (1982). Based on the results obtained by Bonilla (1973) and Matti and others (1982), Ziony and Yerkes (1985) assigned a maximum M6.5-7.0 for non-critical structures near the Sierra Madre fault zone. Similarly, Wensnousky (1986) proposed a maximum earthquake magnitude of 6.4-6.6 along each segment by comparison with the 1971 San Fernando earthquake. The slip rate on the fault is several millimeters per year (Morton and Matti, 1987).
A future M7.7 earthquake that could rupture all seven segments or the whole length of the Sierra Madre fault cannot be excluded. The 19 km-long surface rupture of the 1971 San Fernando earthquake and the two separate 4 km-long aftershock distributions of the 1991 Sierra Madre earthquake are seismological evidence for the existence of the surficially mapped segmentation of the fault zone at depth (Hauksson, 1994). Furthermore, the San Fernando and Cucamonga segments, which have relatively short repeat times, appear to behave at least in some cases as independent segments (Bonilla, 1973; Matti and others, 1982). Thus, the seismological and geological evidence for segmentation along the fault suggests that 6.4-6.6 earthquakes occur more frequently and that a M7.7 earthquake may be a very rare event.

Whittier Fault. The Whittier fault is a right-lateral strike-slip fault with slip rates of 2-3 mm/yr (Gath and others, 1992). It is 74 km long and extends from the north end of the Elsinore fault into the eastern Los Angeles basin (Wesnousky, 1986). The Whittier fault has had up to 30 km of right-lateral slip since 4-9 Ma (Lamar, 1990). The vertical offset on it is more difficult to quantify but is much less and averages about 1.0-1.5 km (Wright, 1991). Although the Whittier fault has not had a large earthquake during historic times, it could generate a M7-7.3 earthquake (Wesnousky, 1986).

Santa Monica and Hollywood Faults. The Santa Monica and Hollywood faults form the north edge of the Los Angeles basin. They appear to have low slip rates, and almost no background seismicity is associated with these faults. The low slip rate of 0.04-0.4 mm/yr (Ziony and Yerkes, 1985) leads to long return times between events. Because these faults can be traced for a distance of more than 100 km, they are thought to be important in hazard evaluations for the Los Angeles Basin (Ziony and Yerkes, 1985). The expected magnitude estimate is beset with large uncertainties. These faults have also been interpreted to be old overturned normal faults that have been reactivated as thrust faults (Davis and others, 1989). Thus some of the old geological offsets may not be representative of the current geological slip rate.

Elysian Park Fold and Thrust Belt. The importance of the Elysian Park fold and thrust belt as a deep-seated seismogenic structure was not fully recognized until the time of the 1987 Whittier Narrows earthquake. It extends along the east and north flanks of the Los Angeles basin for a distance of approximately 100 km (Figure 5). The thrust and fold belt was located by mapping of fold axes and thrust-faulting earthquakes (Davis and others, 1989; Hauksson, 1990). The thrust or reverse focal mechanisms are mostly located to the south of the north-dipping reverse faults such as Santa Monica and Hollywood faults and form broad clusters along the flanks of the basin. Most of the folding of the basin sediments also occurs along the flanks of the basin. This spatial coincidence of the folding and thrust faulting can be explained with fault-bend or fault-propagation folding models (e.g., Suppe, 1985), where the thrust-faulting earthquakes at depth are associated with folding of the cover sequence. The 1987 Whittier Narrows earthquake caused coseismic uplift in the Whittier Narrows region demonstrating how the folding and faulting are causally related (Hauksson and Jones, 1989; Lin and Stein, 1989; Davis and others, 1989).

Southeast of Whittier Narrows, thrust-faulting focal mechanisms delineate a zone of thrust faulting dipping 25-35° north. It is not possible to tell, based on hypocentral distribution and focal mechanisms alone, if this zone consists of one continuous fault or many small abutting or overlapping faults. The zone is closest to the surface at 4-6 km depth, near Yorba Linda. The Elysian Park fold and thrust belt also extends to the west of Whittier Narrows, beneath downtown Los Angeles, and continues into Santa Monica Bay. It may be offset or segmented by the northern end of the Newport-Inglewood fault (Hauksson, 1990). Slip rates determined both from geoldesy and geology suggest the Elysian Park system may be one of the fastest moving in southern California (e.g., ???) at 3-10 mm/yr. Segmentation of this belt is poorly understood and needs further study to adequately constrain the expected magnitude of earthquakes within it. In
addition, more geological studies of Quaternary deformation and geodetic surveys are needed to confirm the fault slip rates.

Torrance-Wilmington Fold and Thrust Belt. The 60-km-long Torrance-Wilmington fold and thrust belt, like the Elysian Park belt, is defined by fold axes and thrust focal mechanisms and extends from offshore Newport Beach, across the Palos Verdes Peninsula, into Santa Monica Bay (Figure 5) (Hauksson, 1990). The structure of this system is more complex and less understood than the Elysian Park. In Santa Monica Bay the two thrust belts merge into a north-dipping imbricate thrust zone. The Shelf Projection anticline in Santa Monica Bay is probably a surface expression of this imbricate thrust zone (Hauksson and Saldivar, 1989). Unlike most other faults in the Los Angeles basin, the Torrance-Wilmington fold and thrust belt dips to the south. It may be a back thrust, that commonly appear at the later stages of thrust fault formation (J. Suppe, personal communication, 1989).

Davis and others (1989) called the primary concealed fault beneath the Palos Verdes Peninsula fault A. Using the retrodeformable cross section technique, they argued that fault A has a slip rate of 1.9-3.45 mm/yr, significantly higher than those estimated for either the Palos Verdes or Newport-Inglewood faults. The Torrance-Wilmington fold and thrust belt needs further studies of the long-term slip rates and segmentation of the concealed faults to determine the recurrence interval and expected magnitude more accurately.

Faults of the Ventura Basin. The frontal fault of the Transverse Ranges extends westward from the Sierra Madre fault to form the Oak Ridge and San Cayetano faults of the Ventura basin. These faults bound the Ventura basin to the north and south (Figure 2). The slip rates on these faults are the highest in the Transverse Ranges (6-12 mm/yr (Yeats, 1988)) and the shortening across the basin has been extremely rapid. Geologic evidence suggests the present-day rate of north-south shortening of the basin is 20 mm/yr (Yeats, 1983). Donnellan (1991) found 6-9 mm/yr of shortening across the Ventura basin from geodetic measurements over 2.7 years and inferred significantly more deformation over a full earthquake cycle. The faults are long enough to produce earthquakes of magnitude 7-7.5 and move fast enough to be a significant hazard to southern California. Recurrence intervals may be as short as 250 years (Yeats, 1988).

Faults of the Eastern Transverse Ranges. The eastern Transverse Ranges extend from the San Andreas fault on the west to the Little San Bernardino mountains on the east (Figure 2). Unlike the western and central Transverse Ranges, where reverse faults are common, the eastern Transverse Ranges are dominated by the San Andreas fault and its many strands that bound the region to the south (Matti and others, 1985). Most of the earthquakes in the eastern Transverse Ranges are associated with the San Andreas fault system as described above. The largest earthquakes off the main fault are both associated with the Landers (M7.3) event in the Mojave desert. The 1992 Big Bear earthquake (M6.4) in the San Bernardino mountains was an aftershock to the Landers event and occurred on a northeast-striking left-lateral strike-slip fault that did not come to the surface (Hauksson and others, 1993). The 1992 Joshua Tree earthquake (M6.1) occurred on a north-striking fault in the Little San Bernardino Mountains, about 10 km northeast of the San Andreas fault and is considered a preshock of the Landers earthquake. The faults of the Joshua Tree and Landers earthquake lie along strike of each other and the Landers earthquake caused an expansion of the Joshua Tree earthquake’s aftershock zone (Hauksson and others, 1993).

The microseismicity of the eastern Transverse Ranges occurs at a very high rate (Hileman and others, 1973; Allen, 1965) and has been extensively studied (e.g., Webb and Kanamori, 1985; Nicholson and others, 1986; Jones, 1988). The shallow seismicity (≤14 km depth) tends to be spatially clustered, with the clusters distributed more or less uniformly over the area. Earthquakes at depths greater than 14 km occur only south of a line connecting the San Bernardino and Banning strands of the San Andreas fault, indicating that it may be a fundamental tectonic
boundary (Jones and others, 1993). This includes some of the deepest earthquakes in southern California. The 1986 North Palm Springs earthquake (see the San Andreas fault section above) occurred at the boundary of the area of deep seismicity (Jones and others, 1986).

Mojave Desert

The Mojave Desert is a large wedge-shaped tectonic block in the central portion of southern California. It is bounded on the northwest by the left-lateral Garlock fault, on the southwest by the right-lateral San Andreas fault. The most active faults are a group of at least seven right-lateral, strike-slip faults that cut through the center of the block on a northwest strike, collectively called the Eastern California Shear Zone (Dokka and Travis, 1990a). Individually these faults all have low slip rates of <1.0 mm/yr (Dokka, 1983; Dokka and Travis, 1990b) but the total strain rate of the zone from geologic and geodetic data is about 8 mm/yr (Sauber and others, 1986; Savage and others, 1990). This strain has been interpreted as part of the relative plate motion, channeling slip from the ridge in the Gulf of California to the opening of the Basin and Range (Dokka and Travis, 1990b; Hauksson and others, 1993).

No Quaternary faults have been identified in either the westernmost or easternmost portions of the Mojave block. This pattern is reflected in the seismicity of the area (Figure 3) with seismic activity occurring in a diffuse, patchy swath, coincident with the band of late Cenozoic faults, that separates the very quiet areas to the west and east. A majority of the earthquakes occur either near the south edge of the block, near the San Bernardino Mountains or in a less active zone in the center of the block that includes the epicenter of the Manix earthquake of April 10, 1947 (M6.5). The background seismicity does not appear to delineate the mapped faults. Focal mechanisms of the larger events generally indicate slip on more northerly striking planes than those mapped at the surface (Figure 4).

The largest earthquake known in the eastern California shear zone is the 1992 Landers earthquake (M7.3) (Hauksson and others, 1993; Sieh and others, 1993). The Landers earthquake sequence resulted from right-lateral strike-slip faulting within a broad zone extending 95 kilometers along five major and many minor faults. The rupture trends northward along its southern section and northwestward on its central and northern sections. From south to north, the five principal faults of the mainshock are the Johnson Valley, Landers, Homestead Valley, Emerson and Camp Rock faults. Coseismic offsets ranged from 2 m to a maximum of 6 m, the largest surficial strike-slip dislocation of the twentieth century in the Western Hemisphere (Sieh and others, 1993). This earthquake clearly demonstrated that the lack of continuous faults does not preclude large earthquakes because many strands can rupture in one event.

Some of the moderate sequences have not been clearly associated with surface faults. The Manix earthquake of 1947 (M6.5) caused 1.6 km of surface rupture on the east-west striking Manix fault (Figure 2; Richter, 1958). Based on the alignment of the larger aftershocks, Richter (1958) suggested that the event was on a concealed northwest-striking structure at depth; however, relatively inaccurate hypocentral locations in that time make interpretation difficult. The aftershocks of the Homestead Valley sequence delineated a cruciform pattern with one line between two mapped faults, both of which showed surface rupture (Hutton and others, 1980). This trend had a more northerly strike than the mapped structures, and the spreading aftershock zone eventually “crossed over” into adjacent blocks with no apparent regard for the bounding faults. The perpendicular lineation may have represented rupture on conjugate faults.

Sierra Nevada and Southern Basin and Range

North of the Garlock fault, which bounds the Mojave Desert on the north, lie the Sierra Nevada and Owens Valley. The Sierra Nevada are a high mountain range underlain by a Mesozoic granitic batholith. The eastern front of the Sierra Nevada is controlled by a normal fault that
is the western edge of the Basin and Range Province (e.g., Wallace, 1984). The Owens Valley, east of the Sierra Nevada frontal fault, is a graben of the Basin and Range. At the southern edge of Owens Valley is the Coso geothermal field, a site of Quaternary volcanism (e.g., Duffield and others, 1980).

The Garlock fault is a left-lateral strike-slip fault. It extends at least 265 km eastward from the San Andreas fault. The large lateral offsets across it, that vary along strike from approximately 64 km in the west to near zero in the east (Smith, 1962), may contribute to the large deflection in the San Andreas fault in southern California (e.g., Hill, 1982). Geologic evidence shows that large prehistoric earthquakes have occurred on the fault (e.g., Burke, 1979; McGill, 1992), with average recurrence intervals of about 1000±500 years (McGill, 1993). McGill (1992) determined a slip rate of 11±mm/yr for the central Garlock fault making it the third most active fault after the San Andreas and San Jacinto faults.

The largest earthquake recorded on the Garlock fault since 1932 had a magnitude of M5.7. It occurred 11 days after the 1992 Landers (M7.3) earthquake and may have been triggered by that event (Hill and others, 1993). Astiz and Allen (1983) showed that small to moderate earthquakes align along the western half of the Garlock fault where aseismic creep is recorded but that seismic activity is very sparse along the eastern half. These earthquakes show left-lateral strike-slip motion.

One of the largest earthquakes in California (M ~ 8) occurred in 1872 on the Owens Valley fault just east of the Sierra Nevada frontal fault (Townley and Allen, 1939; Richter, 1958; Figure 2). Details of this earthquake are sketchy, but geologic evidence suggest that it resulted from several meters of right-lateral oblique normal slip (Lubetkin and Clark, 1985). The largest earthquake within the Sierra Nevada was the 1946 Walker Pass earthquake (M6.0) (Chakrabarty and Richter, 1949) which was notable for its large foreshock (M5.5) and rich aftershock sequence.

At the microseismic level, the southeastern Sierra Nevada and southern Owens Valley are very active. The seismicity of the Sierra Nevada is characterized by intense swarms with no large earthquakes (Jones and Dollar, 1986). These swarms form a lineation parallel to but about 15 km east of the Kern River fault (Figure 3), called the southern Sierra lineation (Jones and Dollar, 1986). The Kern River fault appears to have been inactive for at least 3 million years (Moore and DuBray, 1978). The adjoining section of the Basin and Range, including Owens Valley and the Coso geothermal area, is also characterized by a high rate of swarm activity without an obvious spatial pattern.

The southern Sierra seismic lineation extends southward to join with the aftershock zone of the 1952 M7.5 Kern County earthquake. It resulted from left-lateral, oblique reverse slip on the White Wolf fault (e.g., Stein and Thatcher, 1981) (Figure 2). Aftershocks of the 1952 earthquake are still occurring and are almost the only seismic activity in the southern San Joaquin Valley. The transition between the oblique reverse faulting in the 1952 aftershock zone and the normal faulting in the southernmost Sierra has not been clearly delineated.

Earthquake Potential

The rate of earthquake occurrence has not varied significantly in the last 60 years (Hutton and Jones, 1993). The magnitude-frequency distribution of the southern California earthquake declustered catalog (excluding aftershocks) from 1932 to 1990 has a b-value is 0.74±0.12 (Figure 6). Declustered catalogs have lower b-values than catalogs with foreshocks and aftershocks included because the smaller events of the clusters have been preferentially removed (e.g., Frolich and Davis, 1992). The rate of occurrence of the earthquakes over the last 60 years and the magnitude frequency distribution can be used to estimate the probability of occurrence of
earthquakes of various sizes in southern California. The average repeat time, $T$, for different magnitudes can be estimated from the magnitude frequency distribution as shown in Table 1. If the rate of occurrence of an earthquake of $M \geq M_r$ is $l = 1/T$, then the probability of that earthquake occurring in a time interval, $t$, is $P(t) = 1 - \exp(-lt)$.

The data are dominated by smaller events so the actual rate of large earthquakes in southern California is not well resolved with only 60 years of data. However, by extrapolating from moderate to large events, probabilities of large earthquakes can be estimated. The probability of a $M \geq 6$ earthquake somewhere in southern California in the next 30 years is over 99%, the probability of a $M \geq 7$ in the same time is $65 \pm 14\%$, while the probability of a $M \geq 8$ event in 30 years is $18 \pm 9\%$.

Seismic moment, seismicity and slip rate along faults or rates of relative plate motions have been used to compare the historical seismicity with the long-term seismicity predicted by geological slip rates (e.g. Brune, 1968; Molnar, 1979; Doser and Smith, 1982). Hauksson (1992) used a similar approach to evaluate how much long-term strain has accumulated and how much seismic moment has been relieved in earthquakes on faults in the Los Angeles basin (the difference being their earthquake potential). He found that the historic record of seismicity is in reasonable agreement with the earthquake potential of the surficial faults in the Los Angeles region. However, the concealed thrust faults of the Los Angeles region (the Elysian Park and Torrance-Wilmington fold and thrust belts) appear to be accumulating significantly more seismic moment than they have released in the last 150 years.

Summary

The diverse tectonic deformation and high level of seismicity in southern California result from the broad transform plate boundary between the North America and Pacific plates. In historic time, the area has experienced two great earthquakes; one on the San Andreas fault in 1857 (M7.8) and the other on the Sierra Nevada frontal fault in Owens Valley in 1872 (M-8). About half of the historic large earthquakes (M-7.0) have occurred on elements of the San Andreas system, the San Andreas, San Jacinto, Elsinore and Imperial faults, and half on the other faults, including the reverse faults of the Transverse Ranges and the strike-slip faults of the eastern California shear zone. Moderate-sized and small earthquakes occur throughout southern California. In some cases they form broad zones of seismicity that coincide with major Quaternary faults. However, with the exception of the San Jacinto fault, the major faults that accommodate most of the plate boundary slip, such as the San Andreas fault, are seismically quiescent at the microseismic level during most of the interseismic period between very large events. Swarms of earthquakes occur in the Imperial Valley, the Santa Barbara channel, the Sierra Nevada and the southern Basin and Range.

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Figure 1. Schematic map of the North American - Pacific plate boundary in California.
Figure 2. A map of southern California showing the major faults and physiographic regions and the $M_L \geq 6.0$ earthquakes recorded from 1932 through 1993. Historic fault rupture is shown in red. Large arrows indicate the sense and magnitude of plate motion. Some fault names are abbreviated as: ADF = Anacapa Dume fault; BAF = Banning fault; CF = Cucamonga fault; MCF = Mission Creek fault; ORF = Oak Ridge fault; PVF = Palos Verdes fault; RHF = Raymond Hill fault; SDT = San Diego Trough-Bahia-Soledad fault; SFF = San Fernando fault; SMF = Santa Monica fault; SRF = Sierra Madre fault; SSF = Santa Susana fault.
Figure 3. A map of southern California showing all $M_L \geq 1.5$ earthquakes recorded by the Southern California Seismic Network from 1978 through 1993. Earthquakes of $M_L \geq 4.0$ are shown by circles and $M_L \geq 5.0$ are shown by stars.
Figure 5. Faults and significant earthquakes that have occurred in the greater Los Angeles basin are since 1920. Aftershock zones are shaded with cross hatching. Dotted areas indicate surface rupture, including the rupture of the 1857 earthquake along the San Andreas fault.
1984–1993 \( \text{LOG}(N) = 5.33 - 0.96 \pm 0.01 \)

Figure 6. The annual rate of earthquakes recorded in the central part of southern California from 1984 to 1993 versus magnitude. Straight line represents the b-value fit.
SEISMIC HAZARD ASSESSMENT IN THE CENTRAL UNITED STATES

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ABSTRACT

Problems with and approaches to seismic-hazard estimation in the midcontinent of the United States are evaluated by using recent data on stress regime, crustal age and structure, and seismicity of other stable continental regions. Evaluating earthquake hazard in the central U.S. is difficult because of the lack of identifiable seismogenic faults and because of the low rate of seismic activity. Furthermore, the recurrence intervals of large earthquakes are poorly known, in part because of the short historical record that spans only a fraction of the repeat times of these quakes. The seismotectonic regime of the central U.S. is dominated by the Reelfoot rift complex and the associated New Madrid, Missouri, seismic zone. However, there are other major tectonic structures in the region such as the Nemaha ridge, the Midcontinent rift system, and the Wichita-Ouachita orogenic belt; earthquakes generating damaging ground motion (approximately magnitude 5.0 or greater) have occurred in the states of Ohio, Illinois, Oklahoma, Texas, Kansas, Nebraska, Kentucky, Alabama, and Arkansas, as well as Missouri. Opinions vary widely about the best way to delineate seismic source zones in such a diffuse and varied seismotectonic environment. Moreover, detailed paleoseismic or neotectonic data that could improve hazard assessments are extremely sparse in the central United States. The Meers fault scarp in southwestern Oklahoma, with its evidence for Holocene displacement and its lack of background seismicity, highlights a new set of assessment problems. Development of site-specific probabilistic hazard curves are further hampered by the lack of strong ground-motion data and high-resolution attenuation data. We address aspects of the overall seismic-hazard assessment problem for which neotectonic information provides constraints. These include a seismic source zonation for the central U.S. and estimates of maximum possible earthquakes for these zones, especially for the New Madrid region.

INTRODUCTION

There have been numerous attempts to quantify the seismic hazard in the central United States. The three most systematic, comprehensive, and recent were by the U.S. Geological Survey (Algermissen and others, 1982), Lawrence Livermore National Laboratory (Bemreuter and others, 1989), and the Electric Power Research Institute (EPRI, 1986). The USGS study evaluated the whole of the United States while the LLNL and EPRI studies focused on the central and eastern U.S. (east of the Rocky Mountain Cordillera). All these efforts utilized large teams of investigators and required a substantial amount of judgment as to the relative importance of the record of past seismicity versus the seismogenic potential of known geologic and tectonic structures as they are oriented within the regional stress regime. More localized central U.S. seismic-hazard studies have been conducted by Nuti and Herrmann (1978) and Nuti (1979).

For this report, the central United States is defined as the region bounded on the north by Canada, the south by Mexico/
Figure 1. Example of a site-specific seismic-hazard curve showing ground motion (acceleration) plotted against an annual probability of exceedance. This particular curve is for a nuclear power site in Illinois. (After Bernreuter and others, 1989.)

Seismic-hazard estimation includes a number of elements. Where active and capable faults are known and mappable, as in the western U.S., the hazard will depend on the seismic potential, i.e., the activity rate and the largest earthquakes that the fault(s) can sustain. In the central and eastern U.S., active faults are rarely identified, and additional, less direct steps are necessary. The "classical" approach to hazard assessment for the central U.S. involves: (1) delineating seismic source zones based on either seismicity, tectonics, or a combination of both; (2) assigning a frequency-magnitude recurrence relation and a maximum possible earthquake for each source zone; (3) developing regional anelastic attenuation relations and applying them to sites within the study area; and (4) producing a hazard curve by incorporating contributions from all source zones at a specific site. For an individual site, the hazard curve presents an estimate of the probability of exceeding a particular ground-motion parameter, usually peak or sustained ground acceleration; an example is given in Figure 1. The usual style of presentation for a region is a contour map showing the level of ground motion that will not be exceeded within a specified time period (e.g., Algermissen and others, 1982).

For this study, as part of a symposium on applying neotectonics to earthquake risk evaluation, we will emphasize the problems of identifying seismic source zones and assigning source parameters to these zones; this is where neotectonic information is incorporated into the hazard-evaluation process. We do not address the equally important questions of proper probabilistic and statistical modeling of ground motion.

As with seismic hazard, the seismicity and tectonics of the central United States have been the subjects of extensive previous investigations (e.g., Nuttil and Herrmann, 1978; Nuttil, 1979; Van Schmus and others, 1987; Bickford and others, 1986; Hatcher and others, 1987). A detailed and comprehensive reexamination is not included here; rather, our objective is to define the seismicity and large-scale tectonic features in a general sense in order to characterize the problems in seismic-hazard assessments in the region. In our view, the single most difficult problem is estimating the seismic potential of a zone or a crustal structure. Aside from the question of properly delineating the zone, this seismic potential has two components: an estimate of the maximum possible earthquake and an estimate of the frequency of occurrence of moderate-to-large events (magnitude ≥ 5). Both components are essential for hazard estimation, yet quantitative constraints for these parameters are sparse. For the central U.S. where the historical record of seismicity is short, where the character of the crust at seismogenic depths is obscure, and where the earthquake potential of most of the recognized crustal structures is unknown, assessing the seismic potential is based more on judgment than knowledge. In the following we present a brief overview of the region in terms of its crustal composition, tectonics, stress regime, and seismicity. Finally, we return to the question of seismic "judgement" as part of an exercise of seismic zonation of the central U.S.

CRUST

How can the crust of the central U.S. be usefully characterized for assessing seismic potential? To begin, there is little doubt that earthquakes are generated in the upper crust, above the brittle-ductile transition, 20 to 30 km deep. However, in this region, crystalline basement is concealed beneath a veneer of Paleozoic sedimentary rocks. Virtually all large earthquakes that have sufficient data to closely constrain hypocentral depth occur within the igneous and metamorphic rocks of the upper crust, although some faulting as revealed by aftershocks does extend up into Paleozoic strata. Moreover, there is no documented case of surface fault rupture accompanying any earthquake in the central U.S. (The Meers fault in southwestern Oklahoma is a remarkable exception to this rule for a prehistoric earthquake and will be discussed later in this chapter.)

The crystalline crust of the central U.S. is wholly Precambrian in age, with the possible exception of the southern coastal block (e.g., Hoffman, 1988). Classically, this region is divided into Canadian shield and interior platform, which together comprise a collage of at least five crustal elements (Fig. 2), the
products of major Precambrian orogenic episodes, ranging in age from Superior craton nucleation in the Archean (3.8 to 2.5 Ga) to the middle Proterozoic Grenville orogeny (1.1 Ga) (Sims and Peterman, 1986; Hoffman, 1988). Most age determinations of the crust are from drill-hole samples; the principal outcrops of Precambrian rocks (the Superior craton in Minnesota, the Ozark dome in Missouri, the Llano uplift and Van Horn/ Franklin Mountains of Texas, and the Black Hills uplift of South Dakota) are few and isolated.

This representation of a Precambrian central U.S. crust that grew to the south and east via lateral accretion during successively younger orogenies is derived from data only recently available. U-Pb age dating on zircon concentrates from drill cuttings (Van Schmus and others, 1987) is perhaps the most useful technique for applying these data to problems of midcontinent crustal evolution. Reliable dates are obtained from small samples, which (unlike for Rb-Sr or K-Ar dating) can tolerate some minor weathering and/or alteration. A comprehensive evolutionary framework for our study region is developing rapidly.

Figure 2. Age subdivisions of the crust of the central U.S. The ages apply to the crystalline basement that is covered by Paleozoic strata over most of the region north of the Ouachita system and are derived mainly from U-Pb zircon dates from drillhole samples.

Figure 3. Principal tectonic features of the central U.S. Rift zones and sutures are emphasized over shallow crustal or epeirogenic features. Structures identified primarily by geophysical methods (subsurface) are hatched; those with clear geological expression (surface) are shaded.

TECTONICS

North of the Paleozoic Ouachita system, Phanerozoic tectonics had minimal effect on the crust of the central U.S. The interior platform was consolidated into a vast composite craton by about 1,300 Ma. This is not to say, however, that tectonic processes ceased to operate in the region. The most prominent example of this is the Midcontinent rift system (Chase and Gilmer, 1973; Van Schmus and Hinze, 1985; see Fig. 3). It has the strongest gravity signature in the central U.S. consisting of a belt of sharply defined linear positive Bouguer gravity anomalies extending from Michigan to Kansas, with central highs of +60 mgal flanked by lows of −100 mgal. Rocks in the rift system are contemporaneous with those of the Grenville province to the east, raising the possibility that the two are genetically related. Although the origin of the Grenville province is poorly understood, it may represent an ancient continental-collision zone that formed the Midcontinent rift system behind the suture front in response to extensional forces. A present-day analog to this is the Baikal rift zone of central Asia, which lies well north of the India-Asia collision zone.
Figure 3 depicts a number of other primary tectonic features in the central U.S. and categorizes them according to whether they are expressed at the surface (geologically defined) or in the subsurface (geophysically defined). We preferentially emphasized rifts and sutures in this figure because a recent study (Coppersmith and others, 1987; Johnston, 1989) identifies these structures as important features that localize seismicity in the stable interiors of continents.

The Paleozoic Ouachita thrust and fold belt is the major Phanerozoic suture traversing the study area. It is generally interpreted as a continuation of the Appalachian system (Hatcher and others, 1987), but the connections are concealed beneath the Gulf Coastal Plain sediments of Alabama. The Ouachita belt represents the southern boundary of Precambrian North America; it juxtaposes Proterozoic cratonic crust to the north with crust of unknown age and uncertain character (continental or transitional oceanic) to the south (Viele, 1979).

Another possible but less clear continental suture is the New York–Alabama lineament, the eastern boundary of the study area. The crustal structure that produces this aeromagnetic lineament is within Grenville-age crust beneath the Appalachian décollement. It has been interpreted as a major strike-slip fault associated with continental collision (King and Zietz, 1978); alternatively, it may demark the suture between the Grenville crust of North America and an accreted terrane named the Clingman block by Johnston and others (1985) or the Bristol block by Hatcher and others (1987).

Three major failed continental rift complexes oraulacogens intersect the Ouachita belt at high angles: the Delaware aulacogen of west Texas, the southern Oklahoma aulacogen, and the Reelfoot rift complex. All are Eocambrian (575 to 700 Ma) in age (e.g., Gordon, 1988), but at least the Reelfoot rift, and probably the others, experienced additional extension and intrusion during early Mesozoic to Cretaceous time (Brafle and others, 1984). The similarity in ages of formation of these rifts suggests that they formed as perhaps failed arms of triple junctions (the Reelfoot rift may represent more than one) during an episode of late Precambrian continental breakup that preceded the Ouachita–Appalachian orogeny.

Other smaller crustal features or their geophysical expressions might be included in Figure 3 that perhaps could be relevant to earthquake occurrence in stable continental settings. For example, basement uplifts and basins, gravity and magnetic highs and gradients, mafic and felsic plutons, shallow crustal grabens, and faults with a wide range of dimensions have been considered in the literature. A cause-and-effect relationship between these smaller-scale features and seismicity remains tenuous and, therefore, is not promoted here. Local stress concentrations arising from these crustal inhomogeneities may produce moderate-size earthquakes (up to magnitude 5.0 to 5.5), but we contend that the larger, damaging events will be associated with the major crustal features, mainly rifts, shown in Figure 3. In fact, in stable continental regions worldwide, earthquakes exceeding moment magnitude 6.0 are exceedingly rare except in crust that has experienced extensive rifting since the Mesozoic (Coppersmith and others, 1987; Johnston, 1989).

STRESS REGIME

The stress regime—or more accurately, the orientation of the horizontal principal stresses that has the greatest deviation from lithostatic stress—from the contiguous United States has been estimated by Zoback and Zoback (1980, 1989) using earthquake focal mechanisms, in-situ stress measurements, and the orientation of stress-sensitive geologic features. The principal differences between the 1980 and 1989 studies are that, in the more recent study, Zoback and Zoback deleted stress-orientation estimates based on overcoring data or geologic features older than Miocene and included recent wellbore-breakout data. These changes resulted in significant differences in the 1980 and 1989 stress-regime maps in the eastern and western U.S.; however, the stress regime for the central U.S. remained essentially unchanged. This suggests that the stress regime in the central U.S. is remarkably uniform, with the direction of maximum horizontal compression trending from northeast to east-northeast as the region is traversed from northeast to southwest (Fig. 4).

There are some relatively minor exceptions to the simple
The seismicity of the central U.S. is depicted in Figures 5 and 6. Although the orientation of the horizontal deviatoric component of the stress regime in the central U.S. seems to be very uniform, the distribution of earthquakes decidedly is not. Whether one considers total known seismicity ($m_{\text{L}}>3.5$, Fig. 5) or only the larger events ($m_{\text{L}}>5.0$, Fig. 6), nonrandomness is obvious. While it is likely that this two to three-century snapshot of seismicity is inadequate to show the complete detailed pattern, we argue that it is sufficient to establish an inherent high degree of clustering. It follows that physical reasons must exist for the observed clustering of seismic-energy release in the central U.S.

The distribution of earthquakes shows little correlation with provinces of similar crustal age (Fig. 2). However, if only larger events are considered (see Table 1), there is a good correlation with primary tectonic structures (Figs. 3 and 6). Thus, it is probable that the type of feature, its geologic age, and its orientation stress state described above. An extensional stress province is present in the extreme southwestern corner of the study area in Texas and New Mexico, possibly representing a transitional zone between the active extensional tectonics of the Rio Grande rift directly to the west and the stable platform of the central plains. The stress orientation for the basement crust of the southern coastal block (Fig. 2) beneath the thick deposits of coastal plain sediments is unknown. Of course, the magnitude of the horizontal stress deviation from lithostatic conditions at hypocentral depths is not known anywhere in the study region.

This picture of a uniform deviatoric stress state for the central U.S. has several important implications for seismic-hazard estimation. Most, if not all, earthquakes occur in a brittle upper crust, which was assembled and incorporated into continental North America more than 1 b.y. ago. The borders of this region, at all but the northern margin, experienced additional significant tectonism throughout the Paleozoic and into the Cenozoic. Evidence of this Phanerzoic (and the older Proterozoic) activity remains in the form of the primary tectonic features of Figure 3. At present, and probably since the Miocene, this ancient scarred crust is being subjected to a compressive, regionally uniform stress regime that originates from plate-margin interactions remote from the region itself. Our task now is to use this understanding of stress regime and crustal structure to explain the observed seismicity of the central U.S. and, ultimately, to derive useful estimates of the pattern and severity of future seismic activity.

SEISMICITY

The seismicity of the central U.S. is depicted in Figures 5 and 6. Although the orientation of the horizontal deviatoric component of the stress regime in the central U.S. seems to be very uniform, the distribution of earthquakes decidedly is not. Whether one considers total known seismicity ($m_{\text{L}}>3.5$, Fig. 5) or only the larger events ($m_{\text{L}}>5.0$, Fig. 6), nonrandomness is obvious. While it is likely that this two to three-century snapshot of seismicity is inadequate to show the complete detailed pattern, we argue that it is sufficient to establish an inherent high degree of clustering. It follows that physical reasons must exist for the observed clustering of seismic-energy release in the central U.S.

The distribution of earthquakes shows little correlation with provinces of similar crustal age (Fig. 2). However, if only larger events are considered (see Table 1), there is a good correlation with primary tectonic structures (Figs. 3 and 6). Thus, it is probable that the type of feature, its geologic age, and its orientation
within the prevailing contemporary regional stress regime are all important contributing factors to earthquake generation in stable continental interiors.

The most pronounced cluster of activity (Figs. 5 and 6) centers on the confluence of the Mississippi and Ohio Rivers at the head of the Mississippi embayment and is clearly spatially associated with the Reelfoot rift complex of Figure 3. No earthquake exceeding magnitude 6 has occurred in the central U.S. outside of this zone since settlement of the region by Europeans. (The 1931 West Texas event, moment magnitude 6.3 [Doser, 1987], occurred in a zone of active faulting associated with the Rio Grande rift and thus has a closer affinity to western continental interiors.

The great New Madrid earthquakes of the winter of 1811–1812, as well as the current seismicity of the zone (Figs. 7 and 8), have been extensively discussed in the literature; we need not repeat those discussions here (see Johnston, 1989, for an overview). Clearly, from Figures 3, 5, and 6, and Table 1, the New Madrid zone, including its probable northward extensions, completely dominates central U.S. seismicity. In fact, it has the highest seismic-moment release rate of any seismic zone in a stable continent region in the world (Coppersmith and others, 1987; Johnston, 1989). Why is the New Madrid region unique, considering that other continental interiors contain numerous primary tectonic structures and are thought to be subject to fairly uniform regional stress regimes?

The answer to the preceding question is not straightforward and requires a degree of speculation or seismic judgement. One possible answer is that, with a much longer record of seismicity, other crustal structures in the central U.S. or in other stable continental regions might be the loci of major earthquakes, le., the assumption of a temporally stochastic pattern of earthquake occurrence is invalid. While we cannot exclude this possibility, we do not favor it and cite the highly stochastic character of the larger seismicity record of China (e.g., McGuire, 1979).

We propose four factors that, combined, make the Reelfoot rift complex especially, perhaps uniquely, susceptible to a high rate of seismicity and the generation of major earthquakes. First, as previously mentioned, it is a major, throughgoing crustal structure. This may be essential to localizing a high strain rate area.

Second, the rift is oriented ideally with respect to the

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**TABLE 1. CENTRAL UNITED STATES EARTHQUAKES M ≥ 5.0**

<table>
<thead>
<tr>
<th>Date</th>
<th>Location</th>
<th>Magnitude</th>
<th>MM_rect</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1987 06 10</td>
<td>38.713/87.954 (SE Illinois)</td>
<td>5.1 m_Lg</td>
<td>VI</td>
<td>Taylor and others, 1989</td>
</tr>
<tr>
<td>1985 01 31</td>
<td>41.642/81.109 (NE Ohio)</td>
<td>5.0 m_b</td>
<td>VI</td>
<td>Nicholson and others, 1987</td>
</tr>
<tr>
<td>1980 07 27</td>
<td>38.19/83.34 (NE Kentucky)</td>
<td>5.2 m_b</td>
<td>VII</td>
<td>Hermann and others, 1982</td>
</tr>
<tr>
<td>1983 11 09</td>
<td>37.99/86.46 (SE Illinois)</td>
<td>5.5 m_Lg</td>
<td>VII</td>
<td>Gordon and others, 1970</td>
</tr>
<tr>
<td>1952 04 09</td>
<td>35.52/87.850 (Central Oklahoma)</td>
<td>5.5 M_e</td>
<td>VII</td>
<td>Gordon, 1985</td>
</tr>
<tr>
<td>1937 03 09</td>
<td>40.47/84.280 (W. Ohio)</td>
<td>5.0 m_b</td>
<td>VI</td>
<td>Nutt and Brill, 1981</td>
</tr>
<tr>
<td>1931 06 16</td>
<td>30.89/104.87 (SW Texas)</td>
<td>5.5 M_e</td>
<td>VII</td>
<td>Doser, 1987</td>
</tr>
<tr>
<td>1925 07 30</td>
<td>35.4/101.3 (N. Texas)</td>
<td>5.2 M</td>
<td>VI</td>
<td>Davis and others, 1989</td>
</tr>
<tr>
<td>1917 04 09</td>
<td>38.10/90.20 (E. Missouri)</td>
<td>5.0 m_b</td>
<td>VI</td>
<td>Nutt and Brill, 1981</td>
</tr>
<tr>
<td>1916 10 18</td>
<td>33.5/86.2 (N. Alabama)</td>
<td>5.3 m_b</td>
<td>VII</td>
<td>Steigert, 1984</td>
</tr>
<tr>
<td>1900 05 26</td>
<td>42.0/89.0 (N. Illinois)</td>
<td>5.0 m_b</td>
<td>VII</td>
<td>Nutt and Brill, 1981</td>
</tr>
<tr>
<td>1900 05 16</td>
<td>50.0/104.0 (U.S.—Canada Border)</td>
<td>5.5 m_b</td>
<td>VI</td>
<td>Horner and Hasegawa, 1978</td>
</tr>
<tr>
<td>1905 08 22</td>
<td>36.8/89.6 (SE Missouri)</td>
<td>5.1 m_b</td>
<td>VI-VII</td>
<td>EPRI catalog, 1986</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Date</th>
<th>Location</th>
<th>Magnitude</th>
<th>MM_rect</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1995 10 31</td>
<td>37.0/89.4 (SE Missouri)</td>
<td>6.2 m_b</td>
<td>IX</td>
<td>Nutt and Brill, 1981</td>
</tr>
<tr>
<td>1991 09 27</td>
<td>38.25/86.50 (SE Illinois)</td>
<td>5.5 m_Lg</td>
<td>VII</td>
<td>Street, 1980</td>
</tr>
<tr>
<td>1982 10 22</td>
<td>35.9/95.1 (E. Oklahoma)</td>
<td>5.5 m_b</td>
<td>VII-VIII</td>
<td>Nutt and Brill, 1981</td>
</tr>
<tr>
<td>1977 11 15</td>
<td>41.0/97.0 (E. Nebraska)</td>
<td>5.0 m_b</td>
<td>VII</td>
<td>Nutt and Brill, 1981</td>
</tr>
<tr>
<td>1975 06 18</td>
<td>40.2/84.0 (W. Ohio)</td>
<td>5.2 m_b</td>
<td>VII</td>
<td>EPRI catalog, 1986</td>
</tr>
<tr>
<td>1967 04 24</td>
<td>39.17/86.30 (NE Kansas)</td>
<td>5.1 m_b</td>
<td>VI-VII</td>
<td>Dubois and Wilson, 1978</td>
</tr>
<tr>
<td>1965 06 17</td>
<td>36.5/89.5 (SE Missouri)</td>
<td>5.3 m_b</td>
<td>VII</td>
<td>EPRI catalog, 1986</td>
</tr>
<tr>
<td>1965 10 08</td>
<td>38.7/89.2 (SW Illinois)</td>
<td>5.1 m_b</td>
<td>VII</td>
<td>EPRI catalog, 1986</td>
</tr>
<tr>
<td>1943 01 05</td>
<td>35.5/90.5 (NE Arkansas)</td>
<td>6.0 m_b</td>
<td>VII</td>
<td>Nutt and Brill, 1981</td>
</tr>
<tr>
<td>1932 06 09</td>
<td>36.5/89.0 (S. Central Illinois)</td>
<td>5.0 m_Lg</td>
<td>VI</td>
<td>EPRI catalog, 1986</td>
</tr>
<tr>
<td>1926 02 07</td>
<td>36.5/89.8 (SE Missouri)</td>
<td>7.4 m_b/8.5 M_e</td>
<td>XII</td>
<td>Nutt, 1983</td>
</tr>
<tr>
<td>1912 01 23</td>
<td>36.3/93.6 (SE Missouri)</td>
<td>7.1 m_b/8.4 M_e</td>
<td>X-XI</td>
<td>Nutt, 1983</td>
</tr>
<tr>
<td>1811 12 16</td>
<td>36.0/90.0 (NE Arkansas)</td>
<td>7.0 m_b/8.3 M_e</td>
<td>—</td>
<td>Street and Nutt, 1984</td>
</tr>
<tr>
<td>1811 12 16</td>
<td>36.0/90.0 (NE Arkansas)</td>
<td>7.2 m_b/8.5 M_e</td>
<td>XI</td>
<td>Nutt, 1983</td>
</tr>
</tbody>
</table>

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**Note:** Table 1 includes earthquakes in the central United States with magnitudes M ≥ 5.0, including those occurring in the states of Illinois, Missouri, Arkansas, and Tennessee. The table also includes events in the states of Ohio, Kentucky, and Indiana, as well as those in the states of Nebraska, Oklahoma, and Kansas. The table is divided into two parts: the twentieth century (1901-1987) and the nineteenth century (1801-1900). The table lists the date, location, magnitude, and reference for each event. The references cited in the table are a mix of peer-reviewed papers and reports, including those by Anderson (1986), Doser (1987), Nutt and Brill (1981), and others.
Figure 7. The frequency-magnitude relation for the New Madrid seismic zone (modified from Johnston and Nava, 1985). The data base combines annualized historical seismicity ($M_t$ 3.8 to 6.2) from Nutli and Brill (1981) and the instrumental seismicity of Fig. 8. Recurrence for events of magnitude exceeding $M_t$ 6.2 is extrapolated.

Figure 8. Instrumental seismicity of the New Madrid seismic zone. Data are from the Central Mississippi Valley Earthquake Bulletin published by Saint Louis University. Magnitudes range from low magnitude 1 to magnitude 5.0; depths range from 23 km to shallow (5.0 km, restricted).

Regional stress regime (Fig. 4) for the ratio of shear-to-normal stress to be maximized on preexisting fault systems. (Note that its active west-northwest segment is a good left-lateral strike-slip representation of the auxiliary nodal plane for the right-lateral strike-slip mechanism of the southwest-trending axial zone [Fig. 8]). Other major structures of Figure 3 tend to strike perpendicular or parallel to the regional stress, yielding a less-than-optimum ratio of shear-to-normal stress.

Third, the major Mesozoic-Cenozoic reactivation of the Reelfoot rift is tectonically relatively young, and its crustal disruption has not had time to heal. This may be the factor that explains the aseismicity of the middle Proterozoic midcontinent rift system.

Fourth, and most speculative, is the observation that the Reelfoot rift complex is saturated with water from the largest of the North American drainage systems. It is a wet seismo-geostructure, and some evidence suggests that this may be an important contributing factor for intraplate earthquake generation (Nava and Johnston, 1984; Costain and others, 1987).

CHARACTERIZATION OF INTRAPLATE SEISMIC SOURCE ZONES

To provide a seismic-hazard evaluation for the central United States, we must confront the problem of defining seismic source zones in a region virtually devoid of identifiable active faulting. We propose as a useful approach a classification of seismic source zones that includes information on the degree of knowledge available to define the zone.

In regions such as the central U.S. that lack identified active faults, the concept of a seismic source zone is in itself an admission of lack of knowledge. Abundant seismologic evidence indicates that shallow nonvolcanic earthquakes are satisfactorily modeled as shear failures on planar or at least tabular features we call faults. A seismic source zone, then, represents a geographic region that is judged to contain at least one and perhaps a collection of faults capable of generating earthquakes. Seismic parameters, principally the frequency-magnitude relation and maximum-magnitude earthquake, are assumed to be homogeneous through-
out the zone. Along plate boundaries and throughout most of the western U.S., seismic source zones can be restricted rather confidently to mapped fault zones, although the presence of unrecognized source zones remains (e.g., the Coalinga earthquake for which the causative fault was concealed by an anticline ridge structure of Pliocene and younger age [Clark and others, 1983]).

In the central U.S., seismic source zones are generally large, a reflection of large uncertainty in their definition. Moreover, in an exercise in which 13 experts were requested by Lawrence Livermore National Laboratory to independently zone the central and eastern U.S., the divergence of the resulting maps was startling, as was the range of criteria that the experts used to delineate the source zones (Bernreuter and others, 1989; Anderson, 1986, Fig. 3). Most weight was given to historical seismicity patterns, with tectonic structure and orientation to the regional stress regime also ranking high in importance, but the emphasis and interpretation of each expert varied greatly.

The classification of intraplate seismic source zones (ISSZs) proposed by Johnston (1987) enables one to define seismic source zones in a systematic manner. This is useful because it helps characterize seismic hazard in these regions while incorporating the current level of uncertainty in the definition of the source zones. As used here, the term "intraplate" excludes all features on which plate contact seismicity occurs or zones directly associated with plate margins in which it is clear that relative plate motions are accommodated, even though slip vectors may not be oriented subparallel to the relative plate motion vector. (Examples of such interplate seismic source zones include actual plate boundaries, subsidiary faults in the San Andreas system, and outer-rise or overriding-wedge earthquakes in subduction zones.) The distinction between interplate and intraplate is most difficult in regions such as south-central Asia or portions of western North America, where plate motion is accommodated over a broad zone. Such distributed plate boundaries are commonly included in the intraplate category.

The intraplate designation can be further subdivided according to whether a region is subject to significant Mesozoic-Cenozoic tectonic activity. If this is absent, we term the region "stable continental interior" (SCI). In SCI regions active surface faulting is rare, and consequently, precision and confidence in delineating ISSZs is limited. Our study area, the central U.S., is a SCI region.

The proposed classification for continental, intraplate, seismic source zones is given in Table 2. All intraplate regions are assigned to one of six categories, depending on known (or unknown) tectonic, geologic, and seismologic characteristics. Categories 1 through 6 (Table 2) imply a step-like transition from abundant data that clearly define an ISSZ (category 1 and 2) to a virtual lack of data for background zones (category 6). In reality, the categories are gradational; as new data are acquired and knowledge improves, seismic sources can be redefined into new, better-constrained ISSZs. One of the primary objectives of seismic-hazard research is to upgrade category 3 through 6 zones (where most continental intraplate ISSZs now would be classified) into category 1 or 2.

### SEISMIC-SOURCE ZONATION

To zone the central U.S. for hazard analysis, we must: (1) delineate individual seismic source zones, (2) assign a maximum credible earthquake to each zone, (3) estimate the rate of seismic activity for each zone, and (4) determine the anelastic attenuation from each zone to sites of interest. Estimating the seismic activity and attenuation are beyond the scope of this study, but we will examine how to approach tasks 1 and 2 for the central U.S.

The previously cited study of Coppersmith and others (1987; see also Coppersmith and Youngs, 1989) that assessed the

<table>
<thead>
<tr>
<th>Category</th>
<th>Name</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (A)</td>
<td>Aseismic</td>
<td>An ISSZ within which there is no known significant seismic activity. Moreover, the region is understood well enough geologically and geophysically to exclude with high confidence the possibility future significant earthquakes.</td>
</tr>
<tr>
<td>2 (SG)</td>
<td>Seismogenic</td>
<td>A specific geologic entity (usually a fault) that can be defined geologically or geophysically and, on which, earthquakes are known to have occurred, or there is evidence of prehistoric earthquakes.</td>
</tr>
<tr>
<td>3 (ST)</td>
<td>Seismotectonic</td>
<td>A clearly defined tectonic feature such as a fault zone, rift, suture, intrusion, etc., with which seismicity is spatially associated, but a clear association with a specific fault or faults is lacking.</td>
</tr>
<tr>
<td>4 (S)</td>
<td>Seismic</td>
<td>A region where seismicity is &quot;enhanced over background&quot; and spatial clustering is evident, but data are insufficient to associate the activity with seismogenic or seismotectonic crustal structures.</td>
</tr>
<tr>
<td>5 (T)</td>
<td>Tectonic</td>
<td>Geologic or geophysical data resolve a crustal feature that else where is known to be associated with earthquakes, but in this case no instrumental, historical, or paleoseismic data exist that suggest the feature has experienced significant seismicity.</td>
</tr>
<tr>
<td>6 (B)</td>
<td>Background</td>
<td>A region with no known significant seismicity or known geologic/tectonic features capable of significant earthquakes, but the data are too poor to exclude their existence with confidence.</td>
</tr>
</tbody>
</table>
worldwide occurrence of seismicity in stable continental interiors (SCI) provides a comprehensive data base that can guide source zone definition and maximum-earthquake selection in the central U.S. To counter the probability that the observational record is neither sufficiently long nor complete, Coppersmith and others (1987) compiled data from magnitude >5.0 earthquakes from all stable continental regions. They found fewer than 20 known events of (seismic moment) magnitude >7.0 in these regions, and the level of seismic activity varies greatly on a continent-size scale. Most large events have been preceded by known historical or instrumental seismicity and have occurred in crust of Paleozoic rather than Precambrian age.

Other findings from the SCI study are applicable to seismic source zonation in the central U.S. They include: (1) a compressive, horizontal, deviatoric stress regime dominates in SCI regions worldwide, producing mostly thrust and strike-slip earthquakes; (2) from a total data set of nearly 800 events, $m_p>5.0$ earthquakes are strongly associated with continental rifts of Mesozoic age and younger, and continental passive margins; (3) the rifted-crust association is even stronger for large earthquakes—those that exceed moment magnitude 7 occur exclusively in zones of Mesozoic-Cenozoic rifting, i.e., passive continental margins (successful rifts) or intracontinental (failed) rifts; and (4) surface fault rupture is extremely rare and has been confidently documented in only 1 percent of the SCI data set (8 occurrences).

Given the information compiled in Coppersmith and others (1987), how should one proceed with seismic zonation in SCI regions? The study imposes a strong constraint on source-zone delineation by limiting large (M>7.0) SCI earthquakes to a few possible tectonic settings. Since a seismic zone must have the same maximum earthquake assigned to the entire zone, boundaries should be based on mapped or geophysically inferred structural boundaries, principally of Mesozoic or younger rifts.

The problem of defining the seismic source zone for maximum New Madrid earthquakes was addressed by Johnston and Nava (1985) in their analysis of recurrence probabilities of such events (Fig. 7). They concluded that, although the crustal elastic-strain storage volume for the 1811-1812 earthquake sequence must far exceed the Reelfoot rift boundaries of Hildenbrand and others (1982), major New Madrid earthquakes will be restricted to the principal fault segments within the boundaries of the rift. These segments are delineated by the concentrated pattern of instrumental earthquake epicenters shown in Figure 8. We conclude that the principal seismicity segments of the New Madrid seismic zone must be separately zoned from the rest of the Reelfoot rift complex because it has a different (higher) maximum earthquake potential.

The study of Coppersmith and others (1987) offers useful guidance in restricting the major M>7 earthquakes of SCI regions to a few locales, but what of the significant hazard contributed by damaging, moderate-magnitude events? Background seismicity (e.g., Fig. 5) is an unreliable, even misleading, guide to where such events might occur—witness the M 5.2 Sharpsburg, Kentucky, earthquake in 1980; the M 5.6 New Brunswick earthquake in 1982; or the M 5.0 earthquake near Cleveland, Ohio, in 1986. We conclude that, while major earthquakes can be localized to certain types of primary tectonic structures, one must allow for the occurrence of magnitude 5.0 to 5.5 events virtually anywhere in the central U.S.

Having examined some of the issues involved in seismic source zoning and maximum-earthquake designation, we now proceed to zone the central U.S. In Figure 9, we subdivide the central U.S. into seismic source zones (SSZ) that are labeled according to the type of data used to define the zone (see Table 2). Two requirements controlled the selection of the SSZs in Figure 9. The most important criterion is that the maximum earthquake must be allowed to occur anywhere within the boundaries of the identified source zone. Because fault dimensions of even the largest midplate earthquakes will likely not exceed 100 km (Nuttli, 1983), the SSZs of Figure 9 obviously do not represent monolithic seismogenic structures; rather they are regions within which structures have similar seismogenic potential. In applying this criterion, we emphasize the maximum earthquake component of seismogenic potential rather than seismic activity rate.

Figure 9. Seismic source zones for the central U.S. The criteria for defining each zone is indicated (see categories of Table 2). The estimated maximum earthquake and zone boundaries are derived from arguments presented in the text. See Figure 10 for detail on the Reelfoot rift-New Madrid seismic zone. Abbreviations: T, tectonic; ST, seismotectonic; S, seismic; B, background; SG, seismogenic; A, aseismic.
The first SSZ selection requirement leads directly to the second: boundaries of identified SSZs should be based primarily on the known or inferred extent of primary tectonic features (Fig. 3). This is a significant departure from the past practice of defining seismic source zones based on the record of historical seismicity.

The maximum earthquake estimated for each SSZ in Figure 9 is based on both the largest known earthquake for the zone and the earthquake record of similar SSZs in the global data base of Coppersmith and others (1987). Note that of all central U.S. seismic source zones, only the Reelfoot rift SSZ has experienced the estimated maximum earthquake in historic times.

We have defined fewer seismic source zones in Figure 10 than some previous studies (e.g., Nuttli and Herrmann, 1978; Berrrenger and others, 1989). This is because we recognize the possibility of a moderately large earthquake \( m_s \) over a very broad background SSZ (category 6, Table 2) based on the worldwide study (Coppersmith and others, 1987) that shows many such events in SCI environments cannot be associated with primary tectonic structures. Thus, our background SSZ combines many seismic source zones that previously had been treated separately (e.g., the Ozark uplift, the Colorado lineament, various intra-continental basins or uplifts). Past seismic activity and the orientation to the regional stress field are additional contributing factors that we considered.

**The Reelfoot rift/New Madrid SSZ**

The Reelfoot rift complex is subdivided into two separate SSZs (Fig. 10): a seismic SSZ (Zone A) and a seismotectonic SSZ (Zone B) (see Table 2). Zone A is delineated on the basis of the linear trends of the numerous small-earthquake epicenters (Fig. 8). The linearity of the pattern suggests that this zone is actually composed of several seismogenic fault segments; these probably last ruptured in their entirety in the great earthquake sequence of 1811-1812. Moreover, seismic-reflection profiles have actually imaged an upper-crust disturbed zone that is coincident with the southwestern arm of Zone A (e.g., Crowe and others, 1985).

Zone B is defined by the geophysically inferred limits of the Reelfoot rift complex. Its borders are the margins of the rift as defined by magnetic and gravity data by Hildenbrand and others (1982) to the south, and by Braile and others (1984) to the north. The geophysical signature of the Reelfoot lobe is much clearer than the Saint Louis and Wabash Valley lobes to the north, but the geophysical data and seismic activity are significant enough that these northern branches should not be ignored in hazard zonations.

The east-west Rough Creek graben zone is included as a fourth lobe by Braile and others (1984). It is clearly a rift-type structure, but we classify it as a "tectonic" SSZ (category 3, Table 2) because it has an associated significant seismic activity. The lack of seismicity is probably related to the fact that its orientation is nearly parallel to the prevailing regional horizontal principal stress. We consider the probability of significant earthquakes \( m_s > 5.5 \) in this zone to be much lower than the rest of Zone B; therefore, we remove it from Zone B on the map in Figures 9 and 10.

We assign as the southern boundary of Zone B the inferred extension of the Ouachita foldbelt beneath the Mississippi embayment. This choice of boundary is not based on hard data. It is unclear that the rift structure of Hildenbrand and others (1982) extends to the foldbelt, but there is no evidence that the rift extends south of the Ouachita belt. Therefore, it seems a logical place to truncate Zone B.

In terms of perceived seismic hazard, the distinction between Zone A and Zone B is important: both the maximum possible earthquake and the seismic activity rate differ substantially for the two subzones. We believe that a great earthquake of \( M_s > 7.0, M_L > 8.0 \) would be restricted to Zone A. A possible, although admittedly qualitative, explanation for this is that the crustal rock of stable continental interiors is normally strong enough to inhibit or confine coseismic rupture propagation; only within the faulted and weakened segments of Zone A can rupture propagate to sufficient dimensions to produce great earthquakes. Thus, we regard the New Madrid Zone A as a special case that is virtually unique in North America, with the possible exception of portions of the St. Lawrence rift valley.

Even though the boundaries of Zone B are fairly well defined by geophysical methods, its maximum-magnitude earthquake is difficult to estimate with any degree of confidence. On the basis of the Coppersmith and others (1987) study, we assign an \( m_s 6.5 \) as the maximum probable event. Low magnitude 6 events have occurred in continental-rift environments currently under compression in Europe (Rhine graben), India (Cambay and Godavari grabens), North America (St. Lawrence rift), Australia (Adelaide geosyncline, Fitzroy trough), and Africa (Sirte grabens). Events larger than \( m_s 6.5 \) have occurred in the St. Lawrence and Sirte regions, but we consider these analogous to New Madrid Zone A events. The assigned maximum earthquake
of $m_{0}$ 6.5 has not been experienced in historic times in Zone B, but the occurrence of similar-magnitude shocks in tectonically similar rift settings worldwide suggests such an event is possible in Zone B.

On the basis of the historical seismicity (Fig. 5) and instrumental seismicity (Fig. 8), significant earthquakes are more likely in Zone B north of latitude 35.5. One could argue for separate zones, but we feel this relies too heavily on the short historical record. Nevertheless, the relatively aseismic nature of Reelfoot rift south of Marked Tree, Arkansas, is an enigma.

Epilogue: The Meers fault

The Meers fault, located in the Oklahoma aulacogen (Fig. 3), represents a probable prehistoric exception to the domination of central U.S. seismicity by the New Madrid zone. Strong geologic evidence now indicates a magnitude 7+ earthquake on this fault within the past 1,100 to 1,400 years (Luza and others, 1987; Ramelli and others, 1987; Madole, 1988). If the fault's dip is subvertical at hypocentral depths, its orientation is favorable for left-lateral strike-slip movement, which is the observed dominant slip component. It has been virtually aseismic throughout the historical past.

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NEW KNOWLEDGE OF NORTHEASTERN NORTH AMERICAN EARTHQUAKE POTENTIAL

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Abstract

Most previous seismic hazard maps for Canada and the U.S. were essentially based on a model that assumes a continuation of the historical pattern and rate of seismicity. During the past decade, significant new information on earthquake potential has become available that will affect hazard estimation in the populated regions of eastern North America. New evidence from global studies of earthquakes in stable continental regions has shown that the larger earthquakes occur through reactivation of relatively young rift faults that break the integrity of the continental crust. This implies that future large earthquakes in northeastern North America may occur in currently quiet portions of the passive continental margin, or on the ancient passive margin that runs along the St. Lawrence valley and southwards under the Appalachians. The impact of such geological associations on the hazard estimates depends critically on the relative weights that are given to "geological" and "historical" earthquake source zones in the hazard model. A 50/50 weight implies significantly greater hazard along currently quiet portions of the Iapetus and Mesozoic rifts than current models.
Introduction

It may seem strange to have a Canadian give the introductory paper to a conference on northeastern United States (NEUS) earthquake risk, but I assure you that earthquakes and their geological environments are no respecters of political boundaries. My co-author, Peter Basham, once remarked that it was very brave of the organizers of a 1989 New York Academy of Sciences conference to invite him, because at such meetings he usually told of large earthquakes in Canada that were going to follow him south in the near future. Despite this, he, and now I, have been invited to talk as various U.S. meetings, perhaps reflecting the bounty of Canadian earthquakes that occur in environments similar to those in the northeastern U.S. (NEUS). In part, this is because Canada occupies 2/3 of the stable craton of North America, and although our earthquake history for much of that area is shorter than the U.S.'s, we have had many more recent earthquakes to study (Table 1). We, in parallel with U.S. geologists, have been moving to understand why the large earthquakes occurred where they did, and hence what future earthquakes should be expected in similar terrains elsewhere.

Firstly a few statistics to set the picture and demonstrate the earthquake magnitude scale. There are about 7500 earthquakes greater than magnitude 4 around the world each year. In eastern North America - east of the Rockies, see Figure 1 - the USGS and GSC typically locate 90 earthquakes larger than magnitude (M) 3 each year; perhaps half of these are felt. Within this area, an average year will see a dozen M≥4 and one or two M≥5, an average decade will see perhaps two M≥6, and the average century, two M≥7 (Fig. 1). For comparison, in the last decade (1982-1992) eastern Canada had ten M≥5 and three M≥6. If we confine ourselves to the NEUS, there were only two M≥5 earthquakes, M5.2 in the Adirondacks in 1983 and M5.0 in Ohio in 1986. These rate statistics present a general statement of the total level of seismic hazard in the region, but say little about how the hazard is distributed across the eastern part of the continent.

The seismicity map (Fig. 2) contains the basic information on the spatial distribution of seismic hazards. One does not need to have any earth science understanding to see that the earthquakes form a halo around a less-active region in the centre of the continent, and that the hazard should be similarly distributed.

Many earlier seismic hazard maps (e.g., Basham et al., 1985 for Canada, Algermissen et al., 1982 for NEUS) included available geological information where appropriate, but of necessity were based chiefly on what was known from the historical seismicity record. The subsequent decade of seismicity has not proved this approach wrong in the U.S., as the Adirondacks and Ohio earthquakes were moderate earthquakes that occurred in areas of recognized hazard. The approach proved less satisfactory in Canada, where the decade 1982-1992 produced a number of significant earthquakes remarkable for their size, location, or effects, including:

- 1985 Nahanni, Northwest Territories, M6.9 and M6.6 (exceeded the expected upperbound magnitude by nearly two units; lies just outside ‘eastern’ North America).
- 1988 Saguaneay, Quebec, M5.9 (very deep focus, area of extremely low previous seismicity, enhanced high frequency radiation equivalent to M6.5).
- 1989 Ungava, Quebec, M6.3 (very shallow, produced the first historical surface faulting in eastern North America).
TABLE 1

SIGNIFICANT EARTHQUAKES IN EASTERN NORTH AMERICA

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Year</th>
<th>M</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Madrid region</td>
<td>1812</td>
<td>8.7</td>
<td>1 largest stable craton eq's</td>
</tr>
<tr>
<td>New Madrid region</td>
<td>1811</td>
<td>8.6</td>
<td>1</td>
</tr>
<tr>
<td>New Madrid region</td>
<td>1812</td>
<td>8.4</td>
<td>largest Arctic earthquake</td>
</tr>
<tr>
<td>Baffin Bay</td>
<td>1933</td>
<td>7.3</td>
<td>27 dead from tsunami</td>
</tr>
<tr>
<td>Grand Banks</td>
<td>1929</td>
<td>7.2</td>
<td>early largest earthquake</td>
</tr>
<tr>
<td>Charlevoix, Que</td>
<td>1663</td>
<td>7</td>
<td>Devasting</td>
</tr>
<tr>
<td>Charleston, SC</td>
<td>1886</td>
<td>6.9</td>
<td>prior M6.6 event</td>
</tr>
<tr>
<td>Nahanni, N.W.T.</td>
<td>1870</td>
<td>6.5</td>
<td>10 km surface rupture</td>
</tr>
<tr>
<td>Ungava, Que</td>
<td>1989</td>
<td>6.3</td>
<td>Quebec/Ontario border</td>
</tr>
<tr>
<td>Charleston, Mo</td>
<td>1895</td>
<td>6.2</td>
<td>Vancouver, Canada</td>
</tr>
<tr>
<td>Timiskaming, Que</td>
<td>1935</td>
<td>6.2</td>
<td>Quebec/Ontario border</td>
</tr>
<tr>
<td>Charlevoix, Que</td>
<td>1925</td>
<td>6.2</td>
<td>might be larger</td>
</tr>
<tr>
<td>Cape Ann, offshore</td>
<td>1755</td>
<td>6.1</td>
<td>shaking equivalent to M6.5</td>
</tr>
<tr>
<td>Franklin L., N.W.T.</td>
<td>1992</td>
<td>6.0</td>
<td>shaking equivalent to M6.5</td>
</tr>
<tr>
<td>Sagueneay, Quebec</td>
<td>1988</td>
<td>5.9</td>
<td>Ontario/N.Y. border</td>
</tr>
<tr>
<td>Giles County, Va</td>
<td>1897</td>
<td>5.8</td>
<td>Ontario/N.Y. border</td>
</tr>
<tr>
<td>Massena/Cornwall</td>
<td>1944</td>
<td>5.8</td>
<td>Ontario/N.Y. border</td>
</tr>
<tr>
<td>Miramichi, N.B.</td>
<td>1982</td>
<td>5.7</td>
<td>Ontario/N.Y. border</td>
</tr>
<tr>
<td>Attica, N.Y.</td>
<td>1929</td>
<td>5.5</td>
<td>western N.Y.</td>
</tr>
</tbody>
</table>

Ordered by decreasing magnitude.

*M Preferred magnitude, from GSC for Canadian earthquakes, and
Figure 1. Magnitude recurrence relationship for eastern North America (polygon indicated on the inset map) computed using completeness years for magnitude (M) ≥4 since 1964, M≥5.5 since 1950, M≥6.5 since 1930, M≥7.0 since 1920, and M≥8 since 1800. Note that the three New Madrid earthquakes fall above the trend, suggesting that their recurrence rate may be lower than the historical period suggests. Shaded area on the inset is the ‘stable continental core’ sub-region discussed in the text, and the earthquakes shown are those on Figure 2.
Figure 2. Map of historical earthquakes for magnitude (M) 6 and greater (all known) and M≥5 (since 1920) for eastern North America. Small, medium and large dots represent M≥5, 6, and 7 respectively. Note that the earthquake record in the Arctic is incomplete. Earthquakes discussed in the text are identified as follows: BB, Baffin Bay 1933; C, Charlevoix 1925; CA, Cape Ann 1755; CH, Charleston 1886; F, Franklin Lake 1992; GB, Grand Banks 1929; M, Miramichi 1982; N, Nahanni 1986; NM, New Madrid 1811/12; S, Saguenay 1988; T, Timiskaming 1935; U, Ungava 1989.
On the other hand, some other recent earthquakes have confirmed historical patterns: 1982 Miramichi, New Brunswick (M5.7 and M5.4), 1990 Mont-Laurier, Quebec (M5.0), and 1992 Franklin Lake, Northwest Territories (M6.0).

This paper updates the overview of Basham (1989), which discussed seven of the earthquake sources of large events in North America and summarized twelve possible causal relations for the earthquakes smaller than M6. In contrast to that paper, which discussed the larger earthquakes on an individual basis, we begin with a geological overview, and discuss how the historical earthquakes can be fitted into a conceptual framework, and then how future earthquakes, occurring within that framework, will affect our views on the distribution of seismic hazard.

Role of earthquake source zones in the computation of seismic hazard

Modern methods of computing seismic hazard start by defining geographic regions known as earthquake source zones. The earthquakes in each source zone are used to calculate the seismicity rate (like Fig. 1), and for computing the hazard we assume that the earthquakes occur randomly throughout the zone. The source zones may be defined on various grounds, for example early source zones merely enclosed clusters of historical earthquakes, but more recently source zones have been defined on purely geological grounds. We term these classes of source zones 'Seismicity' Source Zones (SSZ) and 'Geological' Source Zones (GSZ) depending on the nature of the chief hypothesis that characterizes them. As we believe, the best estimates of seismic hazard are likely to come from hybrid source models that each incorporate various proportions of both seismicity and geological information; however, it is convenient to discuss the end member SSZ and GSZ's, but then to represent the range of opinion as to the correct model in terms of alternative 'seismicity-biassed' and 'geology-biassed' models. Early seismic hazard computation programs were intended to handle only a single set of zones for a particular region, but newer codes such as FRISK88 can handle multiple sets, so that the alternative source zones drawn under different hypotheses can now be incorporated directly into the hazard calculations, and the uncertainty in the choice of source models is carried through into the uncertainty in the final calculation.

Causes of large earthquakes on the "stable" continent.

Earthquakes larger than magnitude 7 (this century: 1929 Grand Banks and 1933 Baffin Bay), and perhaps as large as 8 (at New Madrid in 1811/12) have happened in the middle of the North American plate distant from any plate boundary. These large earthquakes dominate any assessment of hazard, and in computing seismic hazard maps we are faced with three questions: what is the likely rate of occurrence of the larger earthquakes? how big can such intraplate earthquakes be? and can they occur just anywhere in the plate interior?

Figure 1 gave one perspective on the rate of activity. There the entire continental part of the North American plate east of the Rocky Mountains was treated as a single SSZ; that is, the earthquakes are presumed to occur randomly within the area shown in the inset. The rates of
earthquakes can be read off the curve, though as most of the contributing earthquakes came from the ‘halo’, the rates are better thought of as applying to the halo region and not the entire eastern portion of the continent. The upperbound magnitude is reasonably easy to fix, being that of the best estimate for the largest of the 1811/12 earthquakes, but judged by how those earthquakes plot above the trend, their energy release may have been a "once in a millennium" event.

However, the seismicity map (Fig. 2) shows that the distribution of the historical seismicity is non-random, and so geologists and seismologists have sought to rationalize the earthquake locations. In our own early work (Basham et al., 1983, Basham and Adams, 1989) we considered that the entire Atlantic passive margin should be treated as a seismicity source zone and effectively defined a geological source zone centred on the rift-margin faults. Thus we concluded that the continental shelf and slope off the eastern United States might experience earthquakes as large as the 1929 Grand Banks earthquake.

The best current hypothesis, championed by Arch Johnston and a team that worked for Electric Power Research Institute (EPRI), is that most stable continental earthquakes occur through the reactivation of relatively young rift faults that break the integrity of the continental crust. Coppersmith et al. (1987) and Johnston (1989) reached this conclusion from a study of worldwide analogs for the North American continent, and showed that 71% of the seismicity of stable continental interiors was associated with extinct continental rifts or continental passive margins (one-sided rifts). Further, all of the 17 earthquakes of magnitude 7 and larger in their compilation are closely associated with the imbedded rifts or passive margins. Many of these earthquakes appear to be thrust events occurring through compressive reactivation of rift (extensional) faults.

Though the EPRI team were concerned primarily with maximum magnitudes in the eastern U.S., the ‘reactivated rift’ hypothesis provides a good explanation for the location of most large eastern earthquakes, and in hindsight explains the Saguenay earthquake of 1988, which was considered a ‘surprise’. Two rift or extensional structures are important for earthquakes in the northeastern U.S. and southeastern Canada: the Atlantic margin, which was formed by the opening of the Atlantic Ocean in Triassic to Cretaceous times (the modern ‘passive margin’), and the Iapetan paleo-margin along the ancient edge of the Grenville-aged continent formed by the opening of the Iapetus (Proto-Atlantic) Ocean about 600-550 m.y. ago. That ancient rifting left a thinned and weakened continental margin, which during the closing of the Iapetus Ocean about 350 m.y. ago was overthrust by the Appalachian mountain range. A schematic section (Fig. 3) shows the relationships of the tectonic units.

**Geological source zones for eastern Canada and adjacent NEUS**

A philosophy of geological source zones for seismic hazard estimation has been refined by Rus Wheeler of the USGS (Wheeler, 1991; Wheeler and Johnston, 1992; Wheeler, in prep). Briefly, a geological source zone is a large region with a common geological history that distinguishes it from neighbouring areas. It must have faults of the same age and type, a seismicity style that is consistent between seismicity clusters, and be in a single stress province. Then faults of the same age and type are presumed to be potentially seismogenic throughout the zone.
Figure 3. Cartoon cross section across the eastern margin of North America (based in part on a figure by Wheeler, 1991). Line of profile is located on Fig. 4. Stipple distinguishes the main orogenic belts. Stars with letters show representative Canadian earthquakes, coded as in Figure 2. Three letter codes above the section indicate geological source zones described in the text.
We show in Fig. 4 approximate geographical limits for a set of GSZ's relevant to Canada. Because this study was done for Canada, the zone boundaries were extended only far enough into the U.S. to ensure the correct computed hazard in Canada. Relative to Wheeler (1991) and Wheeler and Johnston (1992) we distinguish more source zones, as we briefly describe below (refer also to Fig. 3):

Atlantic Rifted Margin (ARM) zone represents the rifted edge of the North American Continent that was thinned by listric normal faulting prior to the opening of the Atlantic Ocean. The zone extends offshore from the landward limit of significant extension to the edge of the continental crust. Magnitude 7 earthquakes in 1929 and 1933 occurred on the Canadian part of the margin: on the U.S. part one or two earthquakes may have occurred off Boston in the 1700s and in 1992 a M4.8 earthquake occurred off New Jersey (Ebel and Kafka, 1992). Earthquakes in this zone may be tsunamigenic if they disturb unstable sediment accumulations on the continental slope.

Mesozoic Rifted Basins (MRB) zone extends from the landward limit of significant extension on the Atlantic margin to the landward limit of limited Mesozoic extension. Other than the amount of extension involved, the other key difference to ARM is that the MRB lacks the large thickness contrast between continental (25-25 km) and oceanic (5-10 km) crust that occurs in the ARM. That contrast may amplify the stresses acting and cause a higher rate of seismicity in ARM than within MRB. The MRB zone encloses the Triassic basins that exist mostly offshore in the Bay of Fundy and Gulf of Maine, and onshore basins such as the Hartford and Newark basins. Similar basins exist in Virginia and are inferred to exist in the southern Appalachians in Georgia (Heck, 1989). The crust in this zone underwent only limited extension, but the reactivation of the Appalachian thrust faults and the creation of the rift-bounding faults makes them good candidates for reactivation in the present stress field (though the basins themselves have low seismicity, Seeber and Armbruster, 1988). Although convincing evidence does not yet exist, we postulate that the 1886 Charleston earthquake could be due to the reactivation of faults that were formed or reactivated during the early rifting episode.

Northern Appalachians (NAZ) zone extends from the landward limit of Mesozoic extensional faulting to the seaward limit of thinned Grenville crust of the Iapetan passive margin. It comprises the terranes of the Appalachian Orogen (comprised of sedimentary rocks and rootless plutons) that over-rode the passive margin. All earthquakes with known depth are relatively shallow, less than 10 km, the prototypical being the Miramichi earthquake sequence of 1982 (Wetmiller et al., 1984).

Iapetan Rifted Margin (IRM) covers the thinned edge of the Grenville continental crust and is substantially as defined by Wheeler (1991, in prep). These ancient rift faults are being reactivated as thrust faults in the current stress regime, as documented for the Lower St. Lawrence and Charlevoix seismicity clusters by Adams and Basham (1991) and by Wheeler (in prep) for Giles County, Virginia and other U.S. Appalachian seismicity clusters. The prototypical earthquake is Charlevoix 1925, for which the moment magnitude has been revised significantly downwards (to 6.2) by Bent (1992). The great depth extent of seismicity - to 30 km at Charlevoix - and the length of the faults suggests large magnitude earthquakes are possible.
IRM as drawn includes all examples of extension entirely within the stable Grenville-aged continent, even those representing only minor extension, as at Attica, N.Y. and in southern Labrador. In fact, most of the extension (and perhaps therefore most of the large faults capable of being reactivated) lies along the ancient continental margin in a narrow zone which passes through the Straits of Belle Isle, along the St. Lawrence River, and east of the Adirondacks. If we were to redefine the source zone to contain only the former margin faults, the remaining region of normal faulting within the continent would be analogous to the limited extension in the onshore Mesozoic basins (zone MRB) inboard of the Atlantic passive margin. By analogy, therefore we term these ‘Iapetan rift basins’, although as we know, some have moved in extension in post-Iapetan times. We discuss the possible future significance of this redefinition later.

Failed Iapetan Rifts (FIR) represent partial weakness of the craton without much extension. In Canada failed arms extend up the Saguenay Fiord and the Ottawa River; in U.S., the New Madrid rift may be similar. Prototypical earthquakes are Timiskaming 1935 M6.2 and Saguenay 1988 in Canada and New Madrid 1811/12 in the U.S. Fault dimensions and earthquake depths are likely to be similar to IRM.

Stable Craton Core (SCC) represents the part of the continent least affected by Phanerozoic extensional faulting. The zone is described further later.

From a cursory view of Fig. 4 it is evident that though the geological zones divide the continent into seismically active and less inactive zones, the density of earthquake within and among the GSZs varies considerably. This is despite having defined the zones to include the same sort of geological features, of presumed similar potential for activity.

Table 2 compares the earthquake density among the zones. As can be seen, the most active zone is forty times more active on a per unit area basis than the least active. Earthquake density within any one zone varies very greatly however, at least as much as among the zones. To take IRM as an example, the zone comprises a seismically-active central third and seismically less-active eastern and western thirds. The rate of activity in the centre is twenty times that of the ends. With regard to the two less-seismic thirds of IRM, we note that while a redefinition of IRM to exclude the Iapetan rifted basins would remove much of the aseismic regions in southern Labrador (a thin line on Fig. 4 shows the suggested boundary), western Pennsylvania and western New York, some aseismic regions remain along the ancient margin proper.

Even within the central third, the seismicity is not evenly distributed, being concentrated in a number of well-known seismicity zones (Lower St. Lawrence, Charlevoix, Montreal) separated by aseismic regions, and likened to beads on a string. We have argued (Adams and Basham, 1991) that such aseismic regions are likely a temporal artifact of our short human timescale - in the immediate future one or more large earthquakes might fill in part of these regions, and if we were to wait for five or ten thousand years we might well not be able to distinguish the seismic from the aseismic regions. Hence, either the future activity in eastern Pennsylvania might be similar to that in the present St. Lawrence valley, or we are missing some fundamental understanding that would allow us to predict the spatial distribution of earthquakes better.
Figure 4. Geological source zones for southeastern Canada and the adjacent U.S. (modified after Hiscock, student work report) with representative seismicity (M≥3 since 1970, triangles; M≥5 since 1940, stars; M≥6 since 1600, circle-stars). Note that the failed Iapetan rifts (FIR) along the Ottawa River and Saguenay Fiord are indicated by arrows.
A pragmatic approach to using geological source zones

For these reasons, we take a pragmatic approach and say that though we are unwilling to employ the 'pure' GSZ model directly for seismic hazard results, we like some of the results. In particular, we think that regional values for the slope of the magnitude-recurrence rate (slope of the line on Fig. 1) derived for each of the large geological source zones are more reliable than values computed from low-level seismicity in small clusters. This is particularly so where either considerable extrapolation to the larger magnitudes is involved, or where the seismicity cluster is dominated by a single large earthquake of unknown return period, presumed to be longer than the historical period. Furthermore, we believe that the upperbound magnitudes in each GSZ should be constant and applied to each contained SSZ. Explicitly, we believe that a low rate of earthquakes does not preclude just as large a upperbound magnitude as might be inferred in a more active zone.

Dealing with the spatial variability within the geological zones is more difficult. At the moment we prefer to use a model (Fig 5) that incorporates seismicity clusters in the same geological zone together (Fig 5, IRX zone) but does not cover the same large geographic area as the geological source zone IRM. That is, we do not have confidence that our understanding of which geological structures are seismogenic is sufficiently complete enough for geology to completely dominate historical experience.
Figure 5. Seismic source zones for southeastern Canada developed under the 'geological' model. The grid has been suppressed for clarity. The IRX zone (outlined in heavy line) incorporates the important contributions from the Iapetan rifted margin (IRM) and the failed Iapetan rift arms (FIR) geological source zones. Note how the seismicity is clustered; 'seismicity' source zones are defined around each cluster for the alternative seismological model. Note also how the NAZ, MRB, and part of ARM GSZ's have been implemented (numbers 1, 2, and 3).

Figure 6. Trial seismic hazard map for southeastern Canada using a 50/50 mix of seismicity-based and geological-based source zones. Parameter contoured is median value of peak horizontal ground acceleration (%g) at a probability of 10% in 50 years.
Hazard implications

For the purposes of structural design we are interested in the exposure of a structure to ground motions during its lifetime. We commonly talk of exceedence probabilities of 10% in a 50 years lifetime, this corresponds to an annual rate of 0.0021, sometimes mis-represented as a 475 year return period. It may seem that we are extrapolating from a short historical record, 100-300 yrs, to a 475- or 1000-year ground motion event, but in fact we are interested in designing and constructing to withstand ground motions with a low chance of exceedence in the building lifetime, a lifetime that is usually considerably less than the length of the historical record. Some have argued that geological processes operate very slowly within continents, and so the chance of an aseismic part of a GSZ becoming seismically active in the near future is low, and that for the next 50 yrs it is rational to expect a continuation of what has happened in the immediate past (i.e. seismicity will re-occur in historically active clusters). This is more or less supported by the long earthquake history in China. Therefore in general we should expect seismically-active zones to continue being active at about their same level. (An analogy can be made with weather forecasting - a simple yet effective predictor forecasts the same weather as on the previous day).

Nevertheless, our past earthquake history, and the geological understanding implied in the GSZ model, indicate this is not the entire story. Only at Charlevoix have large historical earthquakes repeated; all other historical M>5.5 earthquakes (such as Timiskaming, 1935 and Saguenay, 1988) have been the first of their size at their locality. Under this hypothesis we might consider the clusters of low-level seismicity - such as occur near the 1935 epicentre - as long-delayed aftershocks to large earthquakes that occur randomly in time and irregularly in space within a GSZ. We have not explored this avenue further because the implication that the historical seismicity clusters in fact represent single isolated earthquakes and their aftershocks makes the description of their activity by a standard magnitude-recurrence curve suspect.

Recent work in the field of paleoseismology, which seeks to use the geological record to extend the historical record of large earthquakes, has suggested that some large earthquakes (Charleston, New Madrid, Timiskaming) may have been preceded by prehistorical earthquakes in the past hundreds to thousands of years. However the results are still too patchy to be incorporated fully into hazard analysis. For instance, it was only a few years ago that the conventional wisdom for New Madrid was that large earthquakes like 1811/12 were geologically infrequent (Wesnousky and Leffler, 1992); new evidence now suggests a return period of a thousand years or less (Schweig et al., 1993). The field also needs to be viewed in the context that most researchers have been funded to search for paleo-earthquakes in the vicinity of historical events; if the same effort were expended throughout the eastern U.S., we suspect that many new earthquake sources would be found, such as the vicinity of the Wabush valley earthquake (Obermeier et al., 1991).

The above reasons suggest that we need a combination of the SSZ model and the GSZ model, weighted in such a fashion to preserve the seismicity budget and yet allow for the possibility of large earthquakes in hitherto aseismic regions. The GSC is currently suggesting a 2/3 SSZ + 1/3 GSZ combination, though for some trials we have used 50/50, as we illustrate in Fig. 6. Relative to current zoning maps, the hazard for the less seismic parts of the St. Lawrence valley has increased.
An alternative approach would be to take the worst case of either model, and so incorporate an extra level of confidence in design. This approach does not conserve the seismicity budget and so violates the fundamental requirement for a probabilistic analysis. As such, it is inappropriate for building code purposes, but may have some utility in the design of critical facilities where it can be useful to claim conservatism. In our calculations, taking the worst case has a significant effect only where the GSZ lacks contemporary earthquakes, and it occasionally doubles the strong ground motion to significant levels (however at many sites both the SSZ and GSZ levels are so low that even a doubled strong ground motion is still insignificant): everywhere else the SSZ dominates and is already higher than the GSZ.

Issues and solutions to rare earthquakes in the "stable" craton core

Eastern North America within the halo of seismicity discussed above can be expected to have rare but expectable large earthquakes. We sometimes consider these 'surprise' events, though in places like along the Iapetan rifted margin we hope we have demonstrated that such events should be 'expected'. A more difficult problem is the large event that happens away from any previous seismicity or geological structure identified as active. Such 'rogue' events are unexpected, even by most seismologists. And even after the fact it is by no means clear that they can be explained in a way that might have made them 'expected'.

In the previous part of this presentation we have 'explained' much of the seismicity in the halo around the low-seismicity core of North America. To assess the problem of rogue earthquakes in the stable craton core (SCC) we first defined the core region by eliminating all of the continent that contained rifted margins, imbedded rifts, Paleozoic or younger basins, or hot spot tracks (Fig. 1 inset). Because of the very low seismicity of this region it becomes difficult to estimate the hazard level from the seismicity at the province or state scale, except to say that on a space-averaged basis it is very low. If, however, one seeks low-probability estimates of hazard for critical structures or, in Canada's case, nuclear-waste burial vaults, the uncertainties become large because the estimation requires a considerable extrapolation from the few small earthquakes to the extremely rare damaging earthquakes. One of us has been involved in a study (Fenton and Adams, in prep.) of world-wide analogs of SCC. This work follows closely on Johnston's analysis of Stable Continental Interiors (e.g., Johnston, 1989), but because hey wished a more stringent definition for SCC, Fenton and Adams adopted his philosophy but redid the analysis using the NEIS and ISC earthquake databases.

Fenton and Adams considered that the continental cores of Australia, South America, Antarctica, Greenland, Africa, Siberia, and perhaps India were analogous to the SCC of northern Ontario. Earthquakes passing a stringent "completeness" test in those analogous regions were cumulated into a magnitude-frequency relation like that in Fig. 1, and a rate per million square kilometres of 0.004 p.a. for earthquakes larger than M6 was established. The 1989 Ungava earthquake - the first historical surface-rupturing earthquake in SCC - was an event very similar to the well-known Australian earthquakes and confirmed that Australian-type, shallow surface rupturing earthquakes could indeed occur in SCC. The devastating M6.4 Killari (central India) earthquake of September 29th 1993 is a further example. Earthquakes like these provide good evidence that the size of the largest earthquakes in the stable cratons is not M5 (as supposed for example in computing current hazard maps of Canada) but is about 6½.
Thus almost anywhere in SCC has the possibility of being close to a M>6 earthquake, though under the assumption of random occurrence for any particular place the probability is extremely small, and its contribution to seismic hazard at the probabilities appropriate for buildings is effectively zero. Nevertheless, the rates imply a 5% chance that sometime in the next year a large (M>6) earthquake will occur somewhere in the large area of the continent hitherto considered ‘aseismic’. Its effects will be catastrophic only if it occurs near a populated region.

Future of hazard assessment in Canada and its differences to U.S. practice

The National Building Code of Canada (NBCC) is the only model code produced in Canada and the Geological Survey of Canada is the only agency doing regional hazard mapping. The current code, which will remain in force until the year 2000 is based on peak ground acceleration and peak ground velocity maps (Basham et al., 1985), with the design forces being based on peak velocity but adjusted at low building periods for higher or lower acceleration zones.

The last revisions produced hazard maps in 1982 that were incorporated into the 1985 NBCC. Our present hazard mapping work in Canada is focussed on producing new hazard maps in 1995. These will incorporate some of the geological understanding we have discussed above, will use the additional seismicity data accumulated over the past 15 years, will use the best available strong ground motion relations, and will map both peak and spectral ground motion parameters. These maps will be similar in concept to the recent (1991) NEHRP maps. They will be issued for trial use in Canada, revised in the light of this experience, and proposed for adoption into the year-2000 edition of the National Building Code of Canada.

Although other speakers in this meeting will be talking about the estimation of strong ground motion and the computation of seismic hazard, we would be remiss in not pointing out some of the hazard issues that may persist between Canada and the U.S. All of the Canadian seismometers and most of our strong motion instruments are sited on hard rock sites, and for that reason we prefer to estimate the ground motions on bedrock and then apply amplification factors for soil sites. The current tendency in the U.S. is to estimate for ‘firm soil’ and then apply a discount if the site is on rock or an amplification factor for weaker or thicker soil sites. Either of these approaches should give equivalent results, but be warned that the numbers on the Canadian maps will appear different from those on the U.S. maps.

A second issue of concern is that some U.S. regulations - such as the recent landfill regulations (RCRA subtitle D) and a bridge code - still cast earthquake shaking in terms of peak ground acceleration (PGA) alone, due partly to historical precedent and partly to California experience. As I expect you will hear in the next talks, the peak accelerations from eastern earthquakes can be quite high even for small earthquakes which, because they lack much low frequency energy and have a short duration, have a low damage potential. Thus maps of peak acceleration (or some other high-frequency ground motion parameter such as PSV at 0.3 sec), while appropriate for short, stiff structures, can be poor indicator of overall damage.
A third issue is that the specified probability level for the landfill regulations - 10% in 250 years, or 0.0004 p.a. - is much lower than that specified for buildings and is in fact similar to the probabilities specified in nuclear reactor safety assessments in the U.S. and Canada. Such low probabilities are less of an issue in California, because there the hazard is often dominated by the 150-yr return period earthquake on nearby active faults; in the east, however, the intervals between strong ground shaking are much longer (even near seismic zones with palaeoseismic evidence for repeated activity) and the expected shaking continues to rise appreciably with decreasing probability (Fig. 7). When we combine the last two points, as in the landfill regulations, we have had the result of Canadian engineers trying to apply the U.S. regulations (in the absence of an appropriate Canadian standard) and deriving inappropriately conservative designs - ground motions for a landfill site exceeding the design level for a nearby nuclear reactor.

Conclusions

It is clear from the above discussion that there are still many parts of the seismic hazard mapping process that are uncertain. For a structural engineer, expecting variations of strength of a few percent in a stock steel order, the uncertainties may seem daunting. Though some of the uncertainty in seismic hazard estimation is irreducible, due to random fluctuations in the processes, other uncertainties can be reduced as our understanding improves. The geological basis for earthquake distribution presented here is a step forward, even though it may seem that we are claiming less certainty than before - after all we once thought that maps based only on an extreme-value extrapolation of the historical earthquakes were acceptable (e.g., 1970 NBCC). By incorporating a geological understanding and quantifying the uncertainty there should be less surprise in the location of the next M>6 earthquake in the east; though as have past earthquakes, it will doubtless provide a considerable impetus for further improvement in the hazard estimates.

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NEW KNOWLEDGE OF NORTHERN CALIFORNIA EARTHQUAKE POTENTIAL

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(Paper not submitted in time for inclusion in Proceedings)
ESTIMATES OF SEISMIC SOURCE REGIONS FROM CONSIDERATIONS OF THE
EARTHQUAKE DISTRIBUTION AND REGIONAL TECTONICS IN THE PACIFIC
NORTHWEST

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ABSTRACT

The tectonic and geologic setting of the Pacific Northwest is that of an active subduction zone, and west of the
Cascade volcanic arc, there are three distinct earthquake source regions: 1) at the interface between the Juan de Fuca
and the North American plate; 2) within the subducting Juan de Fuca plate, and 3) within the crust of the overlying
North American plate. The record of historical seismicity in the region has few events of magnitude 7 or greater; all
of these events are thought to be within the subducting portion of the Juan de Fuca or Gorda plates, within the crust
of the North American plate, or offshore of northern California in the flat-lying section of the Gorda plate. This
limited historical record is used for nearly all estimates of the earthquake hazards in the region. Unfortunately, the
distribution of seismicity determined using the modern seismic network is not in accord with where many of the
largest historical earthquakes occurred.

The best understood source zone is for those earthquakes that occur intraplate, within the subducting Juan de Fuca
and Gorda plates. Most of the damaging earthquakes in the historical record of the Pacific Northwest have been in
this zone and knowledge of these events has served as the cornerstone for earthquake hazards assessments in the Puget
Sound region. Generally, these events are thought to be caused by gravitational forces within the plate, and that the
events typically occur where the dip of the subducting plate increases from about 10° to more than 25°. Interpretation
of the current available data, including earthquake hypocenters and geophysical imaging of the subducting plates,
indicates that these events should be expected along the strike of the entire subduction zone. Because of the known
plate geometry most of these events are expected to be at depths of 45 to 60 km and have magnitudes as least as
large as that already experienced in the region (7+). The rate of occurrence of these events is unknown, but in Oregon
and Washington there are 6 events known since 1870 that have estimated magnitudes of 6 or greater.

There are no known earthquakes recorded by modern seismic networks on the interface between the two plates and
in the history of the European population in the area no known large event can be associated with the interface.
However, the late Holocene geologic record of subsidence in coastal intertidal marshes provides evidence consistent
with the occurrence of great subduction-style earthquakes. The available geologic record suggests that the return
period for these events is irregular, varying from as frequent as 100 years to longer than 1200 years; the length of the
coast interpreted to have subsided during a particular event is sufficient to have generated earthquakes of magnitude 8
or greater. Some studies have suggested that the entire coast from northern California to central British Columbia
could fail in a single event of magnitude 9 or greater. Despite the fact that there are many unknown details
concerning intraplate events, the general understanding of large-scale plate processes allows a reliable estimate of the
potential source zone for subduction earthquakes. The source zone for these events is very great, including the entire
coast from the deformation front offshore to about the coastline.

Finally, the source zone for crustal events is very poorly known. The historical record includes events greater than
magnitude 7 in central Vancouver Island, the North Cascades, and offshore Northern California. Until the tectonic
causes of these events are better understood, it is not possible to reliably determine the source zones for these events.
Considerable effort is needed to resolve the geological and tectonic details related to these events.

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INTRODUCTION

The first step in assessing earthquake hazards and risks within a region requires that the nature and distribution of earthquake sources be understood. During the past ten years there has been considerable effort in the Pacific Northwest focused on determining the distribution of earthquake sources and on placing these sources within the regional tectonic setting. Two examples illustrate how successful these studies have been in changing the assessment of earthquake hazards in the Pacific Northwest. First, in 1975, Hopper and others (1975) suggested that the Juan de Fuca plate might be attached to the North American plate, thereby making it unlikely that great thrust earthquakes (of magnitude 8 or larger) would occur on the interface between the two plates. Fifteen years later, most earth scientists routinely accept convergence between the two plates, and as a consequence, most further accept the interpretation that subduction zone earthquakes on this interface will be at least as large as magnitude 8.0 (e.g., Heaton, 1990); some scientists argue for earthquakes greater than magnitude 9.0 (Heaton and Hartzell, 1987). The most compelling evidence that has caused this change in earthquake hazards assessments is found in the intertidal marshes along the coast. At some sites in Willapa Bay on the Washington coast, at least 8 discrete episodes of subsidence of the intertidal marsh surface are recorded in the geologic section of the last 4000 years (Grant and others, 1989). Each episode of subsidence is thought to have been the result of a subduction zone earthquake (Atwater, 1987), thus subduction events appear frequently in the geological record (see Atwater, this volume; Rogers and others, this volume, for more discussion).

As a second example, Perkins and others (1980) issued a revised estimate of the maximum horizontal ground acceleration for Oregon and Washington in which the maximum expected earthquake magnitude was estimated for subregions of the two states. In the southern Washington Cascade Range near Mount St. Helens, the estimate used for the maximum expected magnitude was 5.1. By 1985 the St. Helens zone had been identified, and several studies had concluded that a maximum magnitude event of 6.8 should be adopted for engineering design purposes in the region of the St. Helens zone (Grant and Weaver, in press). Both examples of changes in the assessment of earthquake hazards relied heavily on understanding the regional tectonic framework, and in the second case, the change was nearly completely dependent on seismicity studies.

Because of the subduction zone regime, there are three distinct sources of earthquakes in the Pacific Northwest: 1) crustal earthquakes that occur within the overriding North American plate, 2) intraplate earthquakes that occur within the subducting Juan de Fuca and Gorda plates, and 3) interplate earthquakes that are expected to occur at the interface between the Juan de Fuca (and Gorda) plate and the North American plate (subduction or thrust events). West of the Cascade Range the distribution of earthquake source types reflects a combination of the geometry of the subducting Juan de Fuca and Gorda plates, crustal structure within the overriding North American plate, and tectonic interactions among the North American, Pacific, and Juan de Fuca plates. East of the Cascades it is likely that the earthquake distribution reflects the tectonics and structure of only the North American plate.

Of the three source types, crustal earthquakes in the North American plate and events within the subducting plate (we will refer to these as intraplate events or Benioff zone earthquakes) have been the basis of earthquake hazard assessments for the Pacific Northwest (e.g., Algermissen, 1988). In Washington and Oregon, for example, where the historical record is thought to be complete since the 1870's at the magnitude 6 and greater level (Ludwin and others, in press), there were two events that certainly occurred in the crust and six earthquakes that are either considered or known to have occurred within the subducting plate. One of the anomalies of the Cascadia subduction zone is that there are no large historical earthquakes that are thought to have their source on the interface. In most subduction zones it is this interface that produces the great (magnitude 8+) thrust events like the earthquake that struck the Prince William Sound area of southern Alaska in 1964. Recently, efforts have been made to incorporate at least the possibility of great thrust zone earthquakes into the regional hazard analysis (Algermissen, 1988).

This paper focuses on the extent of the three source regions for the Cascadia subduction zone. In defining the source regions, we have relied on recent compilations of earthquake catalogs for Oregon and Washington, studies of regional seismotectonics, investigations of coastal marsh stratigraphy and determinations of the plate geometry. It is clear that intraplate earthquakes, the most frequently observed of the large magnitude events (6+) in the historical record of Oregon and Washington, are understood well enough that the source region expected to produce events in the future can be specified with great confidence. Despite uncertainty surrounding the details of how and when great subduction zone thrust events may occur on the interface, there is clearly a growing acceptance of the past occurrence of these events. As the general forces that produce these events are understood from comparative studies with other subduction zones, it is possible to define the potential source regions fairly accurately. Finally, because the causes of the large magnitude crustal events in the historical record remain obscure, the portion of western Washington and Oregon which may be subject to large magnitude crustal earthquakes remains uncertain. As noted elsewhere (Rogers and others, this volume; Shedlock and Weaver, 1989), narrowing the uncertainty surrounding the occurrence of great
subduction zone events and determining whether the urban centers in the Puget Sound basin and the Willamette Valley as well as those in the Columbia Plateau are subject to magnitude 7, shallow (<20 km) crustal events will require significant investments in time and effort for new experiments, research and modeling.

Our paper differs from a number of recent review articles on the earthquake distribution in this region in that we provide an overview of the earthquake distribution of the entire Juan de Fuca–North American plate system. Our plots of seismicity combine the reviews of Ludwin and others (in press) who summarized seismicity in Oregon and Washington, Uhrhammer (in press) and Hill and others (in press) who discussed seismicity of northern and central California, respectively. To complete our historical perspective, we combined the catalog of earthquakes greater than magnitude 6 provided by Ellsworth (1990) for California with similar sized events listed by Ludwin and others (in press) for Washington and Oregon and events listed by Rogers (1983a) for southern British Columbia. The scope of our review broadens that of Weaver and others (1990) who discussed the crustal earthquake distribution associated with the Cascade Range from Lassen Peak to Mount Baker in Washington.

Finally, a guide concerning magnitudes in this paper. Unfortunately for most readers, seismologists use several magnitude scales to measure the strength of an earthquake at its source. (Magnitudes are distinct from the earthquake shaking effects at discrete sites that are measured by intensities). The different magnitude scales reflect the fact that most scales are appropriate for only a small portion of the wide range of possible magnitudes. Although we have given the appropriate magnitude scale for particular earthquakes (e.g., local magnitude, ML; body-wave magnitude, Mw; surface-wave magnitude, Ms; moment magnitude, M0; or coda magnitude, Mc) in an effort to avoid confusion we have tried to de-emphasize the differences between the various scales and often use only the term magnitude. Generally, magnitudes greater than 8 are determined using seismic moment, magnitudes greater than about 5 but less than 8 are calculated using either body or surface waves, magnitudes less than about 5.5 usually are determined using a Wood-Anderson type of "local" magnitude, and magnitudes less than about 4.5 tend to be calculated from the coda length of the event. More discussion of the magnitude scales and definitions of seismological terms can be found in the appendix to this volume.

EARTHQUAKE DISTRIBUTION

One of the significant problems in assessing the earthquake hazards of the Pacific Northwest is the discordance between the historical earthquake record and that available from modern seismic instrumentation (from about 1950). The historical record is essential because in Washington and Oregon only the 1965 event near Seattle has been greater than magnitude 6 since the World Wide Standard Seismic Network began recording in 1962; since then in California north of Cape Mendocino all events greater than magnitude 6 have been offshore in the Gorda plate. Thus, one goal of any earthquake hazards assessment program must be to place the historical, larger magnitude earthquakes in the tectonic framework. This task is made more difficult in western Oregon and Washington by the lack of known surface faults with Pleistocene to Holocene offsets (Rogers and others, this volume).

EARTHQUAKES GREATER THAN MAGNITUDE 6

Earthquakes estimated to be larger than magnitude 6 since 1870 are restricted to a relatively small portion of the Pacific Northwest (fig. 1). In plotting the earthquakes in figure 1 we have not attempted to include earthquakes along the offshore ridge system other than in the vicinity of the South Gorda Plate. As the distance between the continental area and the offshore ridges and fracture zones increases it becomes increasingly likely that not all events greater than magnitude 6 will be felt or noted. For the continental areas of the northwestern United States the seismic catalogs are thought to be complete at the magnitude 6 level since 1870 (Ludwin and others, in press; Ellsworth, 1990); in British Columbia the catalog of magnitude 6 events is complete since 1899 (G. Rogers, written communication, 1990).

At the southern end of the Juan de Fuca–North American plate system, these larger events have occurred both onshore and offshore of northern California and to the east of Lassen Peak in the Basin and Range province. Large events are found in the historical record from northwestern Washington and central Vancouver Island; they are notably absent over nearly all of Oregon (fig.1). Within this distribution of earthquakes, the most important events that must be incorporated into hazard assessments are those earthquakes greater than magnitude 7 (eight events total excluding the 2 events in Nevada) and a few events of magnitude less than 7.

The 1872 North Cascades earthquake is generally considered the largest earthquake known in the Pacific Northwest, (Milne, 1956), with an estimated felt area magnitude of 7.4 (Malone and Bor, 1979). It was felt over an area of over 1,010,000 square km, including Washington, northern and central Oregon, northern Idaho, western Montana, and southern British Columbia. The earthquake was followed by an extensive aftershock sequence (Milne, 1956). Study of damage reports suggests that the maximum intensity exceeded VII and may have been as high as IX on the Modified Mercalli (MM) scale (Milne, 1956). Because the available historical data do not unequivocally allow...
Figure 1. Location of earthquakes in the Pacific Northwest with magnitudes estimated to be greater than 6 for the years 1870 to 1990; except for the Cape Mendocino area offshore earthquakes clearly related to the ridge system have not been shown. Earthquake magnitudes are scaled by two symbol sizes, with the small symbols representing magnitudes of 6.0 to 6.9, and the large symbols representing magnitudes greater than 7.0. All earthquakes greater than magnitude 7 and three earthquakes greater than magnitude 6.0 discussed in text have year of occurrence noted. Triangles are Quaternary stratovolcanoes in the Cascade Range.
the location and depth of the 1872 earthquake to be determined, both remain subjects of controversy. The location shown in figure 1 was determined by Malone and Bor (1979) based on the intensity pattern; these authors also summarized other suggested locations of the event. Hopper and others (1982) have suggested a shallow crustal depth based upon intensity contours and the extensive aftershock sequence. All instrumentally located earthquakes (since 1970 near the epicenter proposed by Malone and Bor (1979) are shallower than 25 km.

The 1949 south Puget Sound earthquake (M_g=7.1) is one of five earthquakes (in 1909, 1939, 1946, 1949, and 1965) of magnitude 6 or greater known to have occurred in the Puget Sound basin (fig. 1). The 1949 and the 1965 (M_g=6.5) earthquakes caused significant damage in the Puget Sound region (Murphy and Ulrich, 1951; Nuttli, 1952; Algermissen and others, 1965); in fact these two earthquakes have caused most of the major earthquake damage in the Pacific Northwest. These two events have instrumentally determined hypocentral depths of 54 km and 60 km, respectively (Baker and Langston, 1987; Algermissen and others, 1965). No aftershocks were felt or recorded after the 1949 earthquake; instrumentation available at the time would have detected events larger than magnitude 4.5 (M_g). Similarly, following the 1965 earthquake, no aftershocks were felt, and an examination of seismograms recorded on stations operating within the region failed to identify any aftershocks greater than magnitude 2.5 (M_g). Because these large earthquakes have not produced felt aftershocks, Rogers (1983a) noted that this provided one possible way of estimating whether historical earthquakes were within the crust (where felt aftershocks would be expected) or within the subducting plate.

Large earthquakes occurred in central Vancouver Island in 1918 and 1946. The 1918 event was about magnitude 7 and had an extensive aftershock sequence (Cassidy and others, 1988). The 1946 event of magnitude 7.3 (M_g) occurred in the mid to lower crust (about 15 to 25 km) and had almost no aftershocks (Rogers and Hasagawa, 1978). Because of the sparse population of central Vancouver Island at the times of the earthquakes the value of the damage was not large. As these events are within the crust of the North American plate (Cassidy and others, 1988), they raise serious questions as to the possibility of similar events in the heavily populated urban areas in the Pacific Northwest.

Large earthquakes off the northern California coast have been frequent during the last 120 years (fig. 1) and have done structural damage to onshore facilities. Four events are estimated to have been of magnitude 7 or greater; the largest offshore event of felt-area magnitude 7.3 occurred in 1923 and shaking from this shock damaged a number of chimneys in the small towns north of Cape Mendocino (Toppozada and others, 1981; Ellsworth, 1990). In 1980 a magnitude 7.0 (M_g) event occurred within the Gorda plate, just west of the deformation front (fig. 1). Although there was relatively little damage to structures, a highway overpass collapsed with five people on the bridge; fortunately no one was killed (Simon, 1981). The most damaging earthquake in northern California to date is the 1954 Eureka event (fig. 1). That earthquake was estimated to be of magnitude 6.5 (Ellsworth, 1990); one person was killed and there was considerable non-structural damage.

There are two other notable earthquakes in the historical record. An earthquake estimated to have a felt-area magnitude of at least 6.75 (Toppozada and others 1981) occurred in 1873 near the Oregon-California border (fig. 1). The earthquake was felt from near Portland, Oregon to Sacramento, California, but was felt with MM intensity VIII only in a small area near the estimated epicenter (Toppozada and others, 1981). There is no mention of felt aftershocks accompanying the mainshock in newspaper accounts of the time. This observation suggests that either the event was sufficiently far offshore that any aftershocks were not felt or, that it was deep, possibly within the subducting Gorda plate where aftershocks are not necessarily expected. The second event is the 1936 Milton-Freewater earthquake (fig. 1), the largest known event in eastern Washington. Maximum intensity was reported as MM VII (Coffman and others, 1982), and the magnitude was calculated by Gutenberg and Richter (1954) to be 5.75 (M_g). Noson and others (1988) estimated a magnitude of 6.4 for this earthquake based on the felt area. Because all earthquakes in eastern Washington located since 1970 occurred in the crust, the 1936 event is also assumed to be crustal. Numerous aftershocks were felt.

Losses from earthquakes occurring in the Pacific Northwest are dominated by those suffered during the 1949 and 1965 events. Eight people were killed in the 1949 event (Ulrich, 1949) and six died in the 1965 earthquake (Algermissen and others, 1965). In terms of 1984 dollars, $150 million of damage occurred in 1949; the 1965 event did an estimated $50 million of damage (Noson and others, 1988). The effects of the 1949 and 1965 earthquakes are currently used as the basis for engineering design, emergency response planning, and damage and loss estimates in the Puget Sound region (Hopper and others, 1975). Since the 1975 report was issued, the population of the Puget Sound region has increased by more than 600,000 people and many new buildings have been constructed. More importantly, the 1975 study considered only the effects of deep earthquakes, and did not consider the effects of either a shallow crustal earthquake (like the 1872 event) or the possible effects of a great megathrust earthquake. Thus, there is wide agreement in the scientific community that an updated discussion of the sources, the hazards posed by these sources, and revised estimates of earthquake losses are urgently needed in the Pacific Northwest.
SUMMARY OF INSTRUMENTAL SEISMICITY

Although a few seismic stations have operated in the Pacific Northwest since 1900 (see Rogers, 1983a; Ludwin and others, in press), 1960 is usually assumed as the beginning of the instrumental period for the region. By 1960 the combination of the distribution of seismic stations and population density allowed the reporting and location of earthquakes greater than magnitude 4 (Ludwin and others, in press). Ludwin and others concluded that since 1960, for Washington and Oregon, the earthquake catalog is complete above magnitude 4.0 (Mb); in California the catalog is complete at this magnitude level since at least 1960 (Uhrhammer, in press). Since 1960 the number of seismic stations operating in the Pacific Northwest has greatly increased so that by 1990 much of the region was routinely monitored for earthquake activity less than magnitude 2.5 (Mb). Details of the evolution of the network can be found elsewhere (e.g., Ludwin and others, in press; Weaver and others, 1982, 1990; Rogers, 1983b). The distribution of short-period (1 Hz) seismic stations operating in late 1990 is shown in figure 2. Most of the stations in British Columbia are operated by the Geological Survey of Canada, stations in Oregon and Washington are nearly all operated by the University of Washington, and most of the stations in California are part of the U.S. Geological Survey network. A few stations were installed in southwestern Oregon during the late summer and fall of 1990 by the University of Washington (figure 2).

For a plate-wide perspective of instrumental seismicity, we selected well-located earthquakes that have occurred since 1980 greater than magnitude 2.0 (Mb) and that met the following statistical criteria: calculated hypocentral standard errors of less than ±3 km, at least 5 P-waves used in the solutions, and a RMS of the traveltime residuals less than 0.35 seconds. Most hypocenters have statistics considerably better than these criteria. For Washington and Oregon our hypocentral data are taken from the University of Washington catalog and for California the data are from catalogs maintained by the U.S. Geological Survey. Hypocentral catalogs compiled by the Geological Survey of Canada for British Columbia are incorporated into the University of Washington catalog.

In plotting the combined catalog (fig. 3 and 4), we have subdivided our data into two depth ranges, 0-30 km and deeper than 30 km. Earthquakes greater than 30 km depth are nearly all within the subducting Juan de Fuca plate system (fig. 3). Earthquakes less than 30 km depth are within the North American plate in Washington or Oregon; however, along the northern California coast most of the shallow seismicity is probably within the subducting Gorda plate (Hill and others, in press; Eaton, 1989). Inland from the California coast the shallow earthquakes are within the North American plate.

It is clear that the distribution of both the shallow and deep earthquakes is not uniform across the Pacific Northwest. The deep events are concentrated beneath northwestern Washington and northwestern California, with a sparser distribution of events beneath southwestern Washington and northern Oregon (fig. 3). The lack of deep events beneath the southwestern Oregon coast may result in part from a lack of seismic stations (compare fig. 3 with fig. 2); however, as noted earlier, earthquakes greater than magnitude 4 would have been instrumentally recorded since about 1960. Events of this magnitude would have been felt along the Oregon coast throughout the twentieth century, so there is little doubt that at the magnitude 4 level this portion of the subducting Juan de Fuca and Gorda plates is seismically quiet.

The distribution of crustal earthquakes is likewise not uniform across the Pacific Northwest (fig. 4). Weaver and others (1990) used the changes in the crustal seismicity pattern to divide the Cascade Range and adjacent areas into four regions (termed segments); here we expand those segments westward to the coast. In the northernmost segment, from Mount Baker to Mount Rainier, nearly all of the well-located crustal earthquakes are confined to the region between the eastern Olympic Mountains and the Quaternary stratovolcanoes at the western edge of the North Cascades (fig. 4); there are few events within the North Cascades. The second segment defined by Weaver and others (1990) from Mount Rainier to Mount Hood, is the most seismically active portion of the crust. In southern Washington the St. Helens zone (SHZ) is a particularly prominent north-northwesterly striking alignment of earthquakes. Although there is an hiatus of activity east of the SHZ, in general seismicity continues in a broad zone from the area immediately west of the SHZ into southeastern Washington (fig. 4). The third segment from south of Mount Hood to just north of Mount Shasta is seismically very quiet (fig. 4). This segment has not been monitored continuously, but very few earthquakes were observed during the two years (1980-1982) of continuous operation of a 32-station network or since stations were re-installed in the central Oregon Cascade Range in 1987 (Ludwin and others, in press). There is a marked increase in earthquake activity in the fourth segment between Mount Shasta and Lassen Peak and also along the northern California coast (fig. 4).

CRUSTAL THICKNESS AND PLATE GEOMETRY

The composition of the crust of the Pacific Northwest has been investigated using geologic and geophysical methods, but there are few reversed, high-resolution refraction or wide-angle reflection profiles in the region. Mooney
Figure 2. Regional short-period seismic stations operating in the Pacific Northwest and southwestern British Columbia. Triangles are seismic stations; open circles in southern Oregon represent stations installed during the late summer and early fall of 1990 by the University of Washington.
Figure 3. Distribution of earthquakes greater than 30 km depth, 1980 to April 1990. All earthquake hypocenters have at least 6 P waves used in the solutions, with epicentral errors less than ±3 km, depth errors less than ±5 km, and RMS values of less than 0.35 seconds. The 60 km line shows the approximate easternmost extent of the earthquake distribution at these depths, and is from Weaver and Baker (1988).
Figure 4. Distribution of crustal earthquake activity, 1980 to April 1990. Earthquakes are less than 30 km in depth with location statistics the same for the hypocenters shown in Figure 3. SHZ is the St. Helens zone. Volcanoes are indicated by triangles abbreviated as follows: B is Mount Baker, R is Mount Rainier, S is Mount St. Helens, H is Mount Hood, N is Newberry Volcano, SH is Mount Shasta, L is Lassen Peak. [Figure from Weaver and others, 1990].
and Weaver (1989) summarized the existing studies with a contour map of estimated crustal thickness beneath Washington, Oregon, and northern California (fig. 5). The sparse number of seismic lines emphasizes the need for deep seismic control in many areas. Using this map as a working hypothesis for the configuration of the Moho, there are two noteworthy features.

The first is the pronounced eastward increase in crustal thickness from 16 km at the continental margin to about 40 km beneath the western flank of the Cascade Range. Gravity modeling along two profiles in Oregon and preliminary interpretations of electrical and magnetic data collected by the EMSLAB experiment (EMSLAB Group, 1988) along a profile perpendicular to the northern Oregon coast are consistent with crustal thickening. The contours beneath the Klamath Mountains in southwestern Oregon and northwestern California lack seismic control. Nevertheless, the known thin oceanic crust and thick Cascade Range crust support the general trends represented by the contours.

The second major feature of the crustal thickness map is the presence of thick crust beneath the Cascade Range, the Puget Sound basin, and the Columbia Plateau (fig. 5). Crustal thickness is estimated to be at least 38 km over this entire region, and locally reaches 46 km in the southern Oregon Cascade Range. A gradual eastward thinning of the crust occurs beneath the Basin and Range of southeastern Oregon and northeastern California (fig. 5). The Moho shallows beneath the Okanogan Highlands in northeastern Washington, where a reflection profile (line 5 in fig. 5) has been interpreted as indicating a flat Moho at about 36 km depth beneath most of this province (Potter and others, 1986).

The geometry of the subducting Juan de Fuca and Gorda plates has been partly inferred from the location of earthquakes occurring within the plates; often these events are referred to as Benioff zone earthquakes. In the historical record of large magnitude events, it is these events that are most frequently observed. Disregarding the concentration of earthquakes offshore of Cape Mendocino, since 1870 there are at least six large earthquakes that are either known or thought to be within the subducting plate. These events occurred in 1873, along the coast near the Oregon-California border, and in 1909, 1939, 1946, 1949, and 1965, within the Puget Sound basin (fig. 6). The distribution of intraplate earthquake hypocenters (largely those shown in fig. 3), indicates that the subducting Juan de Fuca plate arches upward beneath southern and central Puget Sound; beneath southwestern Washington, the plate dips to the east-southeast, changing to a northeast dip beneath northwest Washington (Weaver and Baker, 1988). The structure of the arch can be seen in figure 3, where we have indicated the eastern extent of the location of hypocenters at 60 km depth (from Weaver and Baker, 1988). Although much of the shallow plate geometry beneath Oregon is either poorly resolved or unknown, it appears that the average dip of the plate (between the deformation front and 60 km) must be greater in northern Oregon than in Washington.

In Washington and northern California there is general agreement between areas of the thickest crust (>38 km) and areas of crustal seismicity (compare fig. 4 & 5). Clearly, however, there are areas where this association does not hold, particularly from the Oregon Cascades eastward. Thus, the segmentation of the crustal seismicity pattern is probably not directly related to crustal thickness. Unfortunately, the structure of the crust is too poorly known in this region to determine if the composition of the mid-to-upper crust may be fundamental in determining areas of crustal seismicity.

Nor does the segmentation of the crustal seismicity pattern match the variation of the geometry of the Juan de Fuca plate. The crustal seismicity in the Puget Sound area lies above the arch in the Juan de Fuca plate, yet relatively sharp variations in the distribution of the earthquakes argue for the importance of unknown crustal structure rather than variations of the subducting plate geometry. Guffanti and Weaver (1988) noted that the high seismicity segment, between Mount Rainier and Mount Hood, was on the southeastern edge of the plate arch, whereas the seismically quiet segment in Oregon was inland from the section of the Juan de Fuca plate that has an increase in plate dip. However, in suggesting these associations, Guffanti and Weaver (1988) did not incorporate the historical seismicity. In particular, when the 1872 earthquake is considered, it is clear that seismic moment release in the North Cascades dominates all of the crustal seismicity south of the Canadian border (Shedlock and others, 1989).

This last observation points to an important difference between seismicity in Oregon and Washington and that throughout California (including that in the subduction regime). In Oregon and Washington seismicity shows a temporal variation on the scale of at least a few decades (Ludwin and others, in press), whereas in California Hill and others (in press) have noted that a few years of microearthquake monitoring coupled with known mapped faults provides a good representation of the long-term (decades to a few centuries) seismicity pattern.
Figure 5. Contour map of crustal thickness in the Pacific Northwest. The solid lines are reversed refraction profiles (sometimes with other supporting geophysical data); dashed lines are gravity or electromagnetic profiles; dotted lines are crustal reflection profiles; areas enclosed by dashed lines have had estimates of crustal thickness made from seismic array studies. Sources are given by Mooney and Weaver, 1989. [Figure modified from Mooney and Weaver, 1989].
Figure 6. Major earthquakes in Oregon and Washington. Small symbols are events with magnitudes estimated from felt areas to be between 6.0 to 6.9, large symbols are events with magnitudes estimated to be greater than magnitude 7. The 1872 and 1936 events are thought to be crustal, whereas all other events are either known or thought to be within the subducting plate system. [Figure from Ludwin and others, 1991].
ESTIMATES OF THE SOURCE REGIONS

PROBABLE ZONE FOR INTRAPLATE EARTHQUAKES

The geometry of the subducting plate (summarized in fig. 3) allows the occurrence of the large earthquakes in the historical record (e.g., 1949, 1965) to be related directly to the plate configuration (Weaver and Baker, 1988). The T-axis from the focal mechanism calculated by Baker and Langston (1987) for the 1949 south Puget Sound earthquake of magnitude 7.1 (Mw) is oriented to the east-southeast, and the 20_ plunge of the T-axis was shown by Weaver and Baker (1988) to be in good agreement with the plate dip angle determined from the earthquake hypocenters. Therefore, Weaver and Baker (1988) concluded that the 1949 earthquake resulted from down-dip tensional forces within the subducting Juan de Fuca plate, an interpretation consistent with observations for many earthquakes in this depth range in other subduction zones (Isacks and Molnar, 1971). Rogers (1983a) reached a similar conclusion concerning the forces responsible for the 1965 south Seattle earthquake and a smaller event of magnitude 5.1 (Mw) in 1976 off the southeastern end of the Vancouver Island coast. Both events were at a depth of about 60 km and focal mechanisms calculated for both earthquakes were normal faulting with the T axes striking northeast and plunging down-dip (Rogers, 1983a).

Based on the agreement between the dip of the Juan de Fuca plate as inferred from earthquake hypocenters determined from the modern seismographic network and the dip of the T-axes calculated for the larger magnitude historical earthquakes, we believe that we can confidently predict the intraplate earthquake source region for the entire plate (fig. 7). We expect that future large magnitude (~7) intraplate events will occur within the Juan de Fuca plate system in the depth range of the 1949 and 1965 events. Although the depths of these events are considered to be well-known, we have chosen to bracket our source region at a shallower depth. An examination of the University of Washington seismic catalog for the years 1970 through 1990 shows that all of the intraplate earthquakes greater than magnitude 4 are below 45 km and that none have been located deeper than the 1976 event. Therefore, we have used the depth range of 45 to 60 km for our estimate of the probable source region for intraplate events (fig. 7).

We emphasize that this probable source region represents the likely areal extent within which an event may occur; the actual dimensions of the fault area associated with an earthquake of approximate magnitude 7 would be expected to be similar to the 40 km long-fault estimated for the 1949 earthquake (Baker and Langston, 1987). The queried area in southern Oregon represents the region of unknown plate geometry where no intraplate earthquakes have been located either because any events that did occur were not large enough to be detected by the existing seismic network or no events have occurred. We note that the expansion of the existing seismic network (fig. 2) will greatly help to resolve this long-standing question concerning whether this portion of the Juan de Fuca plate is currently truly aseismic. In northern California Benioff zone earthquakes again allow the plate depth to be estimated from the trench eastward to the western edge of the Cascade Range (see Cockerham, 1984; Walter, 1986), so we have shown the probable source region here between the same depth limits as in Washington and northern Oregon.

POSSIBLE SOURCE REGIONS FOR SUBDUCTION ZONE EVENTS

At nearly all convergent margins around the world, large magnitude (8+) earthquakes are known to occur; Cascadia is unusual in that there is no known large earthquake in the historical record. However, recent studies of subduction zone characteristics (Heaton and Kanamori, 1984; Heaton and Hartzell, 1986), crustal strain accumulation in Washington (Savage and others, 1981), crustal earthquakes in southwestern Washington (Weaver and Smith, 1983), and the stratigraphy of coastal marshes along the Washington and Oregon coasts (Atwater, 1987; Atwater, this volume; Grant and others, 1989) have all either concluded directly or inferred that the Cascadia subduction zone should be regarded as capable of generating great interface events.

The geological evidence from the coastal marshes is particularly compelling in terms of arguing unequivocally for the occurrence of past great subduction zone earthquakes. This evidence is multiple buried peat horizons in numerous bays along the coast (see Atwater, this volume). The preferred interpretation is that each buried peat layer represents a previous surface soil horizon that was suddenly submerged during a great earthquake. Subsequently intertidal deposits accumulated on the submerged soils, until the deposits reached a level allowing the development of a new marsh surface. At some sites in Willapa Bay submerged soils have fine-grained sands deposited directly on them. Atwater (1987; this volume) and Grant and others (1989) interpret these sands as being deposited by a tsunami, and because they lie directly on submerged peat layers, they are further interpreted as being locally generated by the same earthquake that produced the subsidence.

To date, the marsh data indicate that the return period of great earthquakes in the late Holocene has been irregular, varying from a return period of less than 100 years to more than 1200 years (Atwater, this volume; Grant and others, 1989). At numerous sites along the coast from Humboldt Bay in California to the Copalis River in Washington,
Figure 7. Schematic map of the probable source region for intraplate, down-dip tensional earthquakes. Large magnitude earthquakes (~7-7.5) are expected anywhere within the shaded region. Question marks indicate areas where there are no earthquakes located within the Juan de Fuca plate and the plate geometry is uncertain. Base map is modified from Ludwin and others (in press).
there is considerable field evidence that the last event occurred about 300 years ago; both carbon-14 dating and tree-ringing dating techniques are in reasonable agreement on this date (Atwater, this volume). There remains much uncertainty over both the repeat time for these great earthquakes and the probable magnitude. At present, both issues require showing synchronous behavior of specific marsh horizons at multiple sites on the coast. However, geologic techniques may not allow synchronous motion for a given marsh horizon ever to be proved, but these techniques may be able to rule out the possibility that the entire length of the subduction zone from Cape Mendocino to central Vancouver Island failed in a single earthquake.

The area where great subduction zone earthquakes occur is not in doubt, in that the shallow-dipping interface between the subducting plate and the overlying plate is the fault plane. The issues for the source area for subduction events relate to whether the entire length of the Cascadia subduction zone generates these earthquakes and the width of the fault that slips. A minimum for the length is provided by Heaton and Kanamori (1984). On the basis of an analysis of plate age and convergence rate used in a regression against the observed magnitude of interface events in other subduction zones, they suggested that in Cascadia an event of about magnitude 8.3 ($M_w$) would be expected given the plate age and convergence rate measured there. Such an earthquake might be expected to rupture a length on the order of 150-200 km along the subduction zone. After comparing a number of additional plate parameters such as offshore bathymetry and gravity and the historical rate of moderate (magnitude 5.7+) earthquakes, Heaton and Hartzell (1986) suggested that the entire length of the Cascadia zone (1100 km), from Cape Mendocino to central Vancouver Island might rupture in one great event of magnitude 9 or greater ($M_w$).

Although, as noted above, all of the marsh data are not completely analyzed, the great extent of the coast where evidence of rapid subsidence has now been documented (fig. 8) makes it likely that very large portions of the coast (at least several 100 to many 100's of km) were involved in some of the earthquakes. The impressive number of sites where multiple buried peat sequences have now been found (fig. 8) with similar dates suggests that the most likely cause of the marsh subsidence is coseismic surface deformation during a great earthquake. Because of the extent of the marsh data along the coast and the arguments of tectonic setting raised by Heaton and Hartzell (1986), we believe that the entire length of the Cascadia subduction zone is capable of generating great earthquakes, even though it is not yet clear that the entire length fails during a single event.

The second point in estimating source areas concerns the width of the rupture perpendicular to the coast. Two models, one in which the rupture is largely offshore and one in which the rupture is largely beneath the continent, bracket the range of probable source widths. In the first model, the rupture extends from the trench down-dip along the interface to a depth of 30-40 km. Because of the plate geometry, this width varies along the subduction zone with a maximum beneath northwestern Washington (~200 km) and narrows to less than 100 km beneath central Oregon and areas further south (fig. 3). If the end of the fault rupture were as deep as 40 km, then beneath Washington a significant portion of the rupture area would be beneath land (fig. 9), whereas if the rupture ends near 30 km depth then the eastern extent of the fault area would be near the coast nearly everywhere. In the second model, in areas like Cascadia that have a very high rate of sedimentation offshore, Byrne and others (1988) have argued that as these sediments are subducted, they allow very poor coupling between the two plates from the trench landward possibly as far as the coast. With this model, the potential source area capable of generating subduction zone interface earthquakes in Cascadia is greatly reduced, consisting approximately of the area from about the coast inland to where the subducting plate begins to dip steeply eastward, perhaps at an approximate depth of 50-60 km (Byrne and others, 1988). Because of the plate geometry, this area is particularly small south of the arch beneath Puget Sound (fig. 3).

In our map of the source region for great subduction zone events, we have illustrated the case where the fault zone width extends from the deformation front offshore to just beneath the coastline (fig. 9). We have chosen this fault width because of deformation modeling results that indicate that it is difficult to explain the pattern of sudden subsidence recorded in the coastal marsh stratigraphy without the rupture area extending offshore (Savage and others, 1981). The shaded pattern in figure 9 illustrates the great source area for these events. Based on the record from other subduction zones, interface events can be expected to occur within the entire area. We have illustrated the case where two earthquakes would fill the entire zone; other possibilities have been discussed by Heaton and Hartzell (1986). To date the geological record from coastal intertidal marshes does not rule out the possibility that the entire length of the subduction zone fails in one very great earthquake. However, the same data do not yet provide unequivocal interpretations of whether the zone might break in a series of smaller earthquakes, of approximate magnitude 8 to 8.5 ($M_w$).

The very great area involved in any subduction zone event does indicate the need for large-scale studies of the properties of the Cascadia subduction zone. For example, greatly expanded regional strain studies would help address the issue of whether the plate boundary breaks in a great single earthquake or in a series of smaller events. A second issue is that of the eastern extent of the fault. Clearly, if the limit of the fault zone for a great subduction event were further westward than the limit shown in figure 9, then it is possible that this would favorably influence shaking effects in the major urban areas.
Figure 8. Location of sites in Washington, Oregon, and northern California where coastal marsh stratigraphy studies have found evidence of sudden subsidence. Compiled from Grant and others, 1989.
Figure 9. Example of source areas for two interplate earthquakes on the shallow, dipping interface. Approximate moment magnitude for each event is given. Other combinations are possible—see text for discussion. Map base is modified from Ludwin and others (in press).
KNOWN SOURCE REGIONS OF LARGE CRUSTAL EARTHQUAKES

The known areas of large magnitude (7+) crustal earthquakes in the North American plate in the Pacific Northwest are limited to central Vancouver Island and the North Cascades (fig. 1). The Vancouver Island events were probably related to the stress regime generated by the interaction of the Explorer plate (at the northern end of the Juan de Fuca plate) with the North American plate (Rogers, 1983b). The cause of the 1872 event remains uncertain in that it occurred in an area with very little contemporary seismicity and no obvious surface fault expression (Malone and Bor, 1979).

The existence of these large crustal events raises the question of whether they might occur within the urban areas of western Washington and Oregon. Unfortunately, the sparsity of known Quaternary faulting (Gower and others, 1985) and the current seismicity distribution does little to answer this question. In the Puget Sound basin the crustal earthquakes do not fall along simple, linear fault zones, but appear to be distributed throughout the crust (fig. 4). Zollweg and Johnson (1989) have recently interpreted a sequence of earthquakes on the western margin of the North Cascades as evidence of a southerly dipping fault zone, the first such zone identified in northwestern Washington. Nevertheless, it remains impossible to infer either the possibility of or argue conclusively against a future magnitude 7+ shallow crustal earthquake in Puget Sound.

Mapped Quaternary faulting in the Puget Sound basin is sparse (Gower and others, 1985) although newly collected shallow reflection data may indicate Quaternary faulting within Puget Sound (Harding and others, 1988). Late Holocene marine terraces provide evidence of abrupt uplift at two locations within the basin, Restoration Point (5 km west of Seattle) and Belfair (45 km southwest of Seattle). Such uplifts may be due to an earthquake or earthquakes (Bucknam and Barnhard, 1989). The available seismicity record in the Puget Sound basin is usually interpreted to suggest that the expected maximum magnitude crustal earthquake is less than that expected in either southwestern Washington or southeastern Washington (Ludwin and others, in press). The observations of uplift on recent Holocene terraces by Bucknam and Barnhard (1989) raises doubts about these earlier interpretations and indicates the need for continued paleoseismic studies in the urban areas.

In contrast to the earthquake distribution in the Puget Sound basin, in southwestern Washington, much of the earthquake activity occurs along the St. Helens zone (SHZ), a right-lateral strike-slip zone that is defined for over 130 km (Ludwin and others, in press; Weaver and Smith, 1983). Two earthquakes greater than magnitude 5 have occurred on the SHZ since 1960; in 1961 a magnitude 5.1 (M7) event occurred south of Mount St. Helens and in 1981 a magnitude 5.5 (M7) event occurred on the SHZ to the north of the volcano. This last event is the largest magnitude crustal earthquake recorded in the Pacific Northwest since the modern seismic network was installed. Mount St. Helens directly overlies the zone where a small (few kilometers) right-stepping offset occurs (Weaver and others, 1987). Several studies have assumed that the magmatic system beneath Mount St. Helens prevents the entire 130 km length from rupturing in a single earthquake (Weaver and Smith, 1983; Grant and Weaver, in press). Grant and Weaver (in press) compared possible source areas along the SHZ north of Mount St. Helens with observations of both fault area and magnitudes calculated from earthquakes on other strike-slip fault zones. As a result of this comparison, Grant and Weaver (in press) concluded that an earthquake in the magnitude range of 6.2-6.8 was the expected maximum magnitude event for the SHZ north of Mount St. Helens.

Our plot of crustal earthquake source areas (fig. 10) shows the regions where these events have occurred in the historical record plus the SHZ and the northern end of the San Andreas system in California. The large area shaded in the North Cascades illustrates the uncertainty in the epicentral position of the 1872 earthquake. Although Malone and Bor (1979) concluded that the event probably was on the northeastern flank of the Cascade Range, other studies (Milne, 1956), have suggested that the event was located near the United States-Canadian border. One problem highlighted by this map is that modern seismicity has no indication of the past large crustal events in central Vancouver Island and the North Cascades.

The map does emphasize the advantage of both accurate locations and an understanding of the seismotectonics responsible for crustal earthquakes, in that along the SHZ it is possible to place a large event on a specific structure, as opposed to having to consider it equally likely that the event may occur throughout a given area. We emphasize that fig. 10 represents a very incomplete assessment of the source regions of large crustal events. Considerable regional geology, local Quaternary studies, and regional-scale strain networks, as discussed by Shedlock and Weaver (1989), will be required to reduce the uncertainty in defining source regions for large crustal earthquakes in the Pacific Northwest.

DISCUSSION

Our definition of the source areas for the three types of earthquakes expected in the Pacific Northwest has relied heavily on the historical and instrumental seismicity with reference to the regional tectonic setting. This is most clearly the case with respect to the area of possible intraplate earthquakes. The existing understanding of the
Figure 10. Known source areas for historical crustal earthquakes greater than magnitude ~6.5; dates give the year of events greater than magnitude 7. The hatched area north of Mount St. Helens represents the segment of the SHZ where Grant and Weaver (in press) suggest a maximum magnitude earthquake in the range of 6.2 to 6.8. Map base is modified from Ludwin and others (in press).
geometry of the Juan de Fuca plate system is sufficient to allow us to argue for events like the 1949 south Puget Sound earthquake occurring anywhere along the length of the subducting plate system. Current interpretations of marine seismic reflection data (Hyndman and others, 1990; Couch and Riddihough, 1989; Snively, 1988) show that everywhere offshore the Juan de Fuca plate is dipping eastward at shallow angles (<10°). Onshore, even in areas where the Juan de Fuca plate is currently seismically quiet, inversion of teleseismic arrivals has been interpreted to show that below depths of 40-60 km the subducting plate is nearly vertical beneath the Cascade Range (Michaelson and Weaver, 1986; Rasmussen and Humphreys, 1988). Furthermore, Crosson and Owens (1987) have suggested that teleseismic waveforms recorded near the central Washington coast are compatible with the hypothesis that the Juan de Fuca plate is continuous (that is not faulted or offset) south of the arch beneath northwestern Oregon. Thus, despite variations in the dip of the upper portion of the plate, the known or interpreted characteristics of the Juan de Fuca plate argues strongly for the existence of a subducting plate capable of producing intraplate tensional stresses along the down-dip axis of the plate (see Spence, 1987 for a discussion of plate stresses). Indeed, if the 1873 earthquake at the Oregon-California border (Figure 1) was an intraplate earthquake as suggested by Ludwin and others (in press), then the currently seismically quiet portion of the subducting plate system experienced the second largest Benioff zone earthquake in historic times.

Whether there is any link between the segmented nature of the crustal earthquake distribution and sources for either subduction zone events or crustal earthquakes is unknown. With respect to crustal earthquakes, the segmentation outlined by Weaver and others (1990) is similar to the division of the Cascade Range suggested by Guffanti and Weaver (1988). These authors used differences in the distribution and composition of late Cenozoic volcanic vents (<5 Ma) to divide the Cascade Range into five segments. These volcanic segments were the same as those suggested here except Guffanti and Weaver (1988) had two segments in northern California. In fact, isostatic residual gravity anomalies (Blakely and Jachens, 1990) and the variations in the volume of Quaternary volcanism (Sherrod and Smith, 1990) in the Cascade Range can be used to divide the Cascade Range into segments identical to those of Guffanti and Weaver (1988). Apparently, the segmentation suggested by the seismicity pattern in the Pacific Northwest must be reflecting the same regional tectonic framework that fundamentally shaped the distribution and volume of late Cenozoic volcanism. The components of this tectonic framework (variations in crustal stress, changes in crustal structure) have left their mark in the middle and upper crust, expressed today by the gravity field.

The link between the regional seismotectonic fabric and late Cenozoic volcanism is likely to be particularly important in assessing the earthquake hazards in the Portland, Oregon, area. A recent study of seismicity in the region around Portland has concluded that there may be a series of en-echelon, strike-slip fault zones southwest of the SHZ (Yelin and Patton, 1991). The Portland Hills fault, a few kilometers west of downtown Portland, shows at least Pliocene offsets (Sherrod and Pickthorn, 1989), and is nearly parallel to the proposed en-echelon seismic zones. The Portland area has a history of moderate magnitude earthquakes (>5+), but the most recent, in 1962, was probably a normal faulting event (Yelin and Patton, 1991). This sense of motion is compatible with the hypothesis that the Portland area is a basin formed by crustal extension, and localized crustal extension may explain the presence of upper Cenozoic basaltic volcanism in the Portland urban area, far west of the Cascade axis. Although currently the relation between contemporary seismicity and the Portland Hills fault zone or the basaltic volcanism is not understood, it is clear that the area warrants continued seismic monitoring to determine more completely the seismotectonic relations and the associated earthquake hazards. Unfortunately, as noted by Ludwin and others (in press), since 1970 crustal seismicity in the Portland area has been sparse compared to higher rates of activity in the 1960's.

The segmentation interpreted from the seismicity pattern may provide additional justification for assumptions that are made for modeling ground shaking from crustal earthquakes. In areas where faults are not well exposed at the surface it is necessary to divide a region into subregions with different source characteristics (e.g., Algermissen and others, 1982). Often these divisions reflect geological boundaries and the maximum magnitude earthquake expected within each subregion. If the seismic segmentation is related to broad regional processes or structures, then this will be a further constraint that can be used in determining subregions for ground motion modeling. In addition, the segmentation may determine boundaries for the use of different modeling techniques. For example, across southwestern Washington and northwestern Oregon, the definition of discrete seismic zones (as earthquakes are located) will likely provide a framework within which maximum magnitude events can be estimated from an interpretation of seismicity patterns, geologic mapping, and crustal structure studies. In northwestern Washington, however, the lack of recognized seismic zones, makes it likely that here it will continue to be necessary to rely largely on an areal approach to hazard estimation. Clearly, a more complete characterization of the crustal structure and regional tectonics is key to understanding the generation of the two widely disparate seismicity patterns found in northwestern Washington and southwestern Washington. A second point concerns the possible segmentation of the
Cascadia thrust interface. If any of the processes responsible for the segmentation observed across the Cascade Range owe their genesis ultimately to the direct interaction of the Juan de Fuca plate system with the North American plate, then perhaps some fundamental rupture length along the coast might reflect this segmentation.

We emphasize that the source regions we have shown for the three earthquake types are part of a major revision of earthquake hazards facing the Pacific Northwest. As noted by Heaton and Hartzell (1987), great subduction zone earthquakes will have shaking effects that are felt over much of western Washington, western Oregon, northwestern California and southwestern British Columbia. The possibility of these earthquakes alone is enough to warrant a thorough re-examination of building codes and earthquake preparedness programs throughout the region. We believe it is critical that this re-examination begin immediately in cities in the Willamette Valley such as Eugene, Salem and Portland because these areas have been outside of earlier efforts undertaken in the Puget Sound region (Hopper and others, 1975) that at least have allowed local governments to be aware of some of the potential problems from earthquakes. The fact that Eugene and Portland have not experienced the larger magnitude earthquakes found in other parts of the Pacific Northwest (Puget Sound, North Cascades, northern California) should not be used as justification for delaying reassessments of hazards. We reiterate the fact that the short-term seismic record in the Pacific Northwest can not be viewed as representative of the long-term risk of large earthquakes. In addition, the fact that we judge the entire portion of the subducting Juan de Fuca plate capable of producing intraplate earthquakes like those of 1949 and 1965 in the Puget Sound basin, coupled with the uncertainty surrounding the extent of the areas where large crustal earthquakes should be expected, provides added urgency for a region-wide re-examination of the hazards posed by earthquakes.

Finally, we note that there may be other potential seismic source zones in the Pacific Northwest that may ultimately be important in earthquake hazard assessments. One zone that might be relevant to earthquake hazards is west of the deformation front, in the nearly flat-lying portion of the Juan de Fuca and Gorda plates. Earthquakes in this portion of plates were referred to as oceanic intraplate by Astiz and others (1988). Off the coast of northern California, this zone is particularly active. But as noted above, oceanic intraplate earthquakes in the historical record have caused relatively little damage to onshore facilities. Nevertheless, as the collapse of the highway bridge during the 1980 Eureka earthquake indicated, these events must be acknowledged in hazards assessments along the northern coast of California. Off the Oregon and Washington coasts only an occasional event is located within the Juan de Fuca plate west of the deformation front; Spence (1989) discusses a few of the known major events. Because there are so few events known off the Oregon and Washington coasts and the tectonic setting of these intraplate events is very uncertain, it is not yet possible to assess how important events west of the deformation front may be in the overall earthquake hazards assessment of the Pacific Northwest. However, if the historical record of damage from this zone from northern California is appropriate for Oregon and Washington, combined with the fact that the distance between the deformation front and the coastline steadily increases northward from Cape Mendocino, then it is likely that oceanic intraplate events will be of considerably reduced importance in most of the Northwest compared with the three zones discussed here.

SUMMARY

In the convergent margin setting of the Cascadia subduction zone, three distinct earthquake sources are possible: 1) earthquakes at the interface between the Juan de Fuca and North American plate; 2) earthquakes within the crust of the overlying North American plate; and 3) earthquakes within the subducting Juan de Fuca plate. For each source type we have defined the approximate region where we expect an earthquake of that type can be expected to occur. The probable source region for intraplate earthquakes within the Juan de Fuca plate is the best known, as we are able to combine the historical data from the 1949 and 1965 earthquakes with the modern instrumental record. The latter data have been used to infer the geometry of the Juan de Fuca plate whereas the former have been used to deduce that the large magnitude earthquakes occur at least in part in response to down-dip tectonic forces within the subducting plate. We suggest that the entire subduction zone, at depths between 45 and 60 km, is capable of producing these events.

Despite many unresolved issues surrounding great subduction zone interface earthquakes, the available geological record, combined with the plate tectonic setting, is interpreted as evidence that these events have occurred in the late Holocene along the coast. Because these events occur on the shallow dipping portion of the plate interface, the general location of the events is well known. Subduction zone events represent a major threat to the population of the Pacific Northwest that has not been integrated into current hazard assessments. A program to accomplish this integration will necessarily have to consider the large scale of the source region for these earthquakes. Finally, the possibility of large crustal earthquakes in the urban areas remains very poorly studied in the Pacific Northwest. Major new initiatives will be required to determine whether the urban centers in western Washington and Oregon must contend with the problems posed by this source type.
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PREDICTION OF GROUND MOTION IN NORTH AMERICA

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Abstract

Because recordings of ground motion are not distributed uniformly across North America, the methods for predicting ground motions are regionally dependent. Most of the recorded ground motions are from the coastal regions of California, and for these regions empirical studies provide the basis for the prediction of ground motion for most earthquakes of engineering interest. These studies show that the response spectra depend strongly on magnitude and site conditions. This means that the scaling of a standard spectral shape by peak acceleration is a poor means of estimating spectral amplitudes. In regions away from coastal California the prediction of ground motion must use a combination of theoretical and observational studies. A particularly useful method, often called the stochastic model, was developed somewhat over ten years ago and has been widely applied in regions lacking ground-motion data from earthquakes of engineering interest. In all regions the near-surface geologic materials can strongly influence the earthquake shaking at the Earth's surface and must be accounted for.

Introduction

The design of any engineered structure is based on an estimate of ground motion, either implicitly through the use of building codes or explicitly in the site-specific design of large or particularly critical structures.

There are a number of methods for estimating ground motions, which we classify into two groups: empirical estimation, making use of previously-recorded ground motions, and estimation that uses seismological models to account for the seismic source, wave propagation en route to the site, and modifications introduced by local geologic structure. Of course, such a division is somewhat arbitrary, for the modeling studies often use empirical data to fix some of the parameters, and the empirical estimates usually are based on regression fits to a functional form suggested by theoretical considerations.

We are concerned in this paper with ground-motion estimation throughout North America. We start the paper with a short discussion of factors that affect ground motions. Specific results are then given for the empirical and theoretical models. We discuss which model is appropriate for various regions in North America; in general, the empirical model can be used to predict ground motions in coastal California, but not in central and eastern North America, where the theoretical model should be used. The theoretical model can also be used for predictions of ground motions in other regions of North America. This
paper will not be a comprehensive survey of ground-motion estimation, on the scale of, for example, Joyner and Boore (1988). Instead, we will concentrate on our recent work, which is applicable to ground-motion prediction from shallow earthquakes. With one exception (Boore, 1986), our previous research has not included the prediction of ground motions from subduction earthquakes such as might occur in the coastal region of the Pacific northwest. For that region, we make a few remarks and direct the reader to the literature.

Parameters Describing Ground Shaking

A number of different quantities calculated from strong-motion records may be used for purposes of seismic design. Peak acceleration is the most commonly used; other quantities used are peak velocity and response spectral values. The response spectrum is defined as the maximum responses, to a given motion, of a set of single-degree-of-freedom oscillators (for example, mass-spring systems) having different natural periods and damping. The response spectral values are useful in structural design because they take account of the frequency of the structure. The response spectrum can be thought of as the maximum responses, to a given motion, of a set of simple mathematical models of structures.

Peak horizontal acceleration may be used in simplified procedures for evaluating liquefaction potential and in pseudostatic studies of slope stability. Peak acceleration has also been commonly used in the past as a scaling parameter to scale a normalized spectral shape and obtain response spectra for analysis of structural response. This is an unsound procedure. It would be valid in general only if the shape of response spectra were independent of earthquake magnitude, source distance, and recording-site conditions. A number of studies (McGuire, 1974; Mohraz, 1976; Trifunac and Anderson, 1978; Joyner and Boore, 1982; Boore et al., 1993), however, have shown that the shapes of response spectra are strongly dependent on magnitude and site conditions. Since long-period ground motion increases with magnitude more than peak acceleration, scaling some sort of average spectral shape by peak acceleration will seriously underestimate long-period motions at large magnitude. At periods greater than about 0.3 sec, large errors can result from the practice of scaling by peak acceleration. These errors can be partially avoided by Newmark and Hall's (1982) method, in which the spectrum at short-periods, intermediate periods (about 0.3 to 2.0 sec), and long periods is derived by applying a scaling factor to peak acceleration, peak velocity, and peak displacement, respectively. Our work, however, shows that the proportionality factor between velocity and intermediate-period response varies significantly with magnitude and site conditions and that the shape of the response spectrum varies significantly with distance; furthermore, the proportionality factors at short periods may not be appropriate for earthquakes in eastern North America. We prefer to estimate response spectra directly, either by regression of individual spectral ordinates for a suite of periods or by using the theoretical model to be described later. One point deserves emphasis: the search for a single parameter to characterize ground motion is doomed to failure. Because the shape of the spectrum changes with magnitude and site conditions, a single parameter that represents ground motion well at one period must necessarily fail to do so at others.
Explanatory Variables

Magnitude

Earthquake ground motions depend on the size of the earthquake, the most common measure of which is magnitude. Reduced to their essence, conventional measures of magnitude are defined in terms of peak motions observed on seismograms from particular instruments after correction for attenuation to a reference distance. The waves radiated from an earthquake are made up of a wide spectrum of frequencies, and seismic instruments provide views into different frequency windows of the radiated energy (some emphasize long-period motions, while others respond to higher-frequency shaking). Because of this, the size of any earthquake can be measured by a number of magnitude scales. There is no guarantee that the magnitudes for any earthquake will agree with one another (nor, according to seismological models of the source, should they be expected to). While confusing, it must be remembered that each scale provides information about the spectral excitation of the source at different frequencies. The most commonly used magnitudes in engineering design are the Richter local magnitude $M_L$, the short-period magnitude $m_{bLg}$, the surface-wave magnitude $M_S$, and the moment magnitude $M$.

$M_L$ is determined from the trace amplitude on a record made by a particular kind of seismograph, the Wood-Anderson seismograph, located within a few hundred km of the earthquake, and is used primarily in California. $m_{bLg}$ is a short-period magnitude scale commonly used in North America east of the tectonic areas of western North America. It is measured from peak motions recorded at distances up to a thousand kilometers on instruments with a passband generally in the 1 to 10 Hz range. The peak motions from these instruments usually correspond to the $Lg$ wave, a relatively slowly-traveling wave made up of $P$ and $S$ waves reverberating in the earth’s crust. $M_S$ is determined from the ground motion associated with surface waves of 20 s period recorded anywhere in the world. The moment magnitude, $M$, is measured from the energy radiated from the source at periods long enough so that all of the higher-frequency complexity of the source is smoothed out. Obtaining $M$ for an earthquake is not as straightforward as it is for $m_{bLg}$. The moment-magnitude scale, first explicitly described by Hanks and Kanamori (1979), is related to the seismic moment of the fault through the simple relation $M = (2/3) \log M_0 - 10.7$; the seismic moment $M_0$, which controls the amplitude of long-period seismic waves, is the product of the average slip across the fault surface, the area of the fault surface, and the modulus of rigidity (or the shear modulus) of the rocks surrounding the fault.

As stated before, a number of magnitudes can be measured for a given earthquake. By combining data from many earthquakes (and, as discussed later, by using theoretical calculations in the case of $m_{bLg}$), empirical relations between the magnitude scales can be obtained. Relations obtained in this way for $M_L$, $m_{bLg}$, and $M_S$ as a function of $M$ are shown in Figure 1. Each of these magnitudes is numerically equal to moment magnitude over only a limited range of magnitude. The nonlinear relations between the magnitude scales is a potential source of confusion and bias in ground-motion prediction, and it is
essential in engineering practice that the magnitude scale being used be clearly stated. We recommend the use of moment magnitude $M$. It corresponds to a well-defined physical property of the earthquake source, and is thus a measure of the size of an earthquake in a very specific sense. Use of moment magnitude has the advantage of making it easier to relate earthquake occurrence rates to geologically determined fault slip rates (e.g., Joyner and Fumal, 1985). It is sometimes stated that because $M_L$ and $m_b \log L$ are determined from instruments with a natural periods in the period range of greatest engineering interest, they should be preferred as the measure of earthquake size to use in making ground-motion estimates for engineering purposes. Catalog values of $M_L$ and $m_b \log L$ for large earthquakes, however, are commonly poorly determined, and moment magnitude is the better measure for such use.

Another argument in favor of a short-period magnitude scale in eastern and central North America is that regional catalogs of earthquake seismicity usually use $m_b \log L$ as the measure of earthquake size, and therefore estimates of seismicity and design earthquakes are specified in terms of $m_b \log L$. Recent methods of ground-motion estimation, on the other hand, use $M$ as the fundamental measure of source strength. It is possible to redo the earthquake catalogs in terms of $M$ by using correlations between $M$ and areas enclosed by contours of equal intensity (Hanks and Johnston, 1992). We recommend that this be done, but until then a relation must be established between $m_b \log L$ and $M$. There are not enough data to establish such a relation, particularly for large earthquakes. The available data are shown in Figure 2, where the squares outline the range of magnitudes published for individual earthquakes, and the filled circles are the magnitudes preferred by the first author of this paper. Overall, there is a good correlation between the two magnitudes. Some of the scatter is due to real differences in the relative amounts of energy radiated at high- and low-frequencies, and some of the scatter is observational error due to limited available data (this is particularly true for the older earthquakes which control the correlation at high magnitudes). The data suggest a linear relation between $m_b \log L$ and $M$. On the other hand, model calculations indicate that the relation should have curvature in the sense that there is less increase in the high-frequency magnitude ($m_b \log L$) for a given increase in the low-frequency magnitude ($M$) for large earthquakes. The relation shown in Figure 1 (which is the same as the solid curve in Figure 2) came from fitting a polynomial to simulated pairs of $m_b \log L$ and $M$ points, computed by using the stochastic model described later in this chapter.

Distance

Because the rupture surface for earthquakes may extend over tens or hundreds of kilometers, there is ambiguity in defining the source distance for a strong-motion record. Various measures of source distance have been used in the development of relationships for estimating ground motion. Some of these are illustrated in Figure 3. The early analyses tended to use epicentral distance because it was readily available. Obvious problems arise with the use of either epicentral or hypocentral distance in the case of earthquakes like the 1966 Parkfield, California, earthquake or the 1979 Imperial Valley, California, earthquake, which have very long rupture zones with the epicenter at one end and recording stations
at the other. For some stations the epicenter and hypocenter are many times more distant than the closer portions of the rupture which are in fact the sources of the peak motions. Similar problems arise with the use of distance to the centroid of the rupture. Some stations may be far from the centroid but close to the rupture. In general it must be expected that different parts of the fault rupture will produce the peak motion at different recording stations. It might seem that the distance measure to use is the distance to the part of the rupture producing the peak motion. Where that part of the rupture is located, however, is unknown for many past earthquakes and for all future earthquakes. Most recent work has used some variation on the closest distance to the rupture. We used the closest distance to the point on the Earth’s surface directly above the ruptured part of the fault (Joyner and Boore, 1981, 1982; Boore et al., 1993).

**Site Conditions**

It is well known that geologic conditions near the Earth’s surface can produce large effects on ground shaking. In many applications the site effect is calculated on a site-specific basis; we will not review the methods for doing so here. A more general accounting for site effects can also be useful, particularly if the predicted motions are for a general class of sites rather than a specific site. In this case, a number of schemes have been used for grouping local sites into a few categories. Some of the groupings are based on descriptive terms (e.g., “rock” and “soil”), while others have a more quantitative basis. We have recently used the time-weighted average of the shear-wave velocity from the surface to a depth of 30 m as the basis for grouping sites into four classes, following the grouping proposed for the 1994 edition of the National Earthquake Hazard Reduction Program’s recommended code provisions. We have also developed equations for site-specific ground-motion predictions, again using the shear velocity averaged over the uppermost 30 m.

**Shallow Earthquakes in Western North America**

Ideally, a sufficient number of recordings of ground motion near a site would be available to allow a direct empirical estimation of the motions expected for a design earthquake. This is rarely possible, as there are simply too few recordings available. Because large earthquakes are relatively rare events and the funding for strong-motion instrumentation is limited, the number of available records is small, particularly for large magnitudes and small source distances, just the conditions most critical for earthquake-resistant design. On the positive side, there are hundreds of ground-motion recordings in western North America for shallow earthquakes, a fortunate situation not shared by other regions of North America. For example, Figure 4 shows the magnitude and distance distribution of recordings used in our recent derivation of prediction equations (Boore et al., 1993). The method for predicting ground motions in western North America (and more specifically, coastal California) is to fit equations or graphical curves to these data, with explanatory variables that include magnitude, distance, and some measure of site condition. A number of different relationships for estimating ground motion have been developed. A comprehensive review of relationships developed before the 1979 Imperial Valley, California, earthquake is given by Idriss (1979).
earthquake marked a major change in the strong-motion data base by providing many more near-source data points than had been available previously. More recent reviews have been written by us (Boore and Joyner, 1982; Joyner and Boore, 1988) and Campbell (1985).

Results for Site Classes

We have recently revised our equations for estimating horizontal ground motion from shallow earthquakes in western North America. In doing this, we classified the sites into groups A, B, C, and D, depending on their shear velocity. The grouping is given in Table 1. The functional form of the equations is given by:

$$\log Y = b_1 + b_2(M - 6) + b_3(M - 6)^2 + b_4r + b_5 \log r + b_6G_B + b_7G_C$$

where $Y$ is the ground-motion quantity to be estimated, $M$ is the moment magnitude of the earthquake, $d$ is the shortest distance (km) from the site of interest to the point on the Earth's surface directly over the fault rupture, and $G_B = 1$ and $G_C = 1$ for site classes B and C, respectively, and are zero otherwise (group D is poorly represented in the data set and was not included in the regression analysis). The values of the coefficients are given in Boore et al. (1993).

The results for site class C as a function of distance are shown in Figure 5. Note that the sensitivity to magnitude is greater for long-period response spectra than it is for peak acceleration and short-period response spectra, a finding consistent with previous studies and expected from seismological models of source scaling. Note also that curves for each magnitude have the same shape. This is imposed by our choice of the functional form (equation (1)), and is in contrast to the assumed shape in several other studies. Studies of residuals of the data with respect to our curves gives us no reason for introducing additional parameters to allow the shape of the curves to be magnitude dependent.

The dependence on magnitude is also given in Figure 6, which shows acceleration response as a function of period for a fixed distance. Clearly, the shape of the response spectrum is a strong function of magnitude, reinforcing the conclusion found in earlier studies that it is inadvisable to construct a design spectrum by simply scaling a fixed spectral shape.

The response spectra are strongly dependent on site class, as shown in Figure 7. Although not easy to see on the linear scale used in the figure, the amplification of sites B and C relative to class A increases with oscillator period; it is more than a factor of 3 for class C sites at periods greater than 1 sec. The dependence on site also exists at the shortest periods in our analysis.

It is important to study the residuals of the data relative to the predictions in order to reveal any biases in the prediction equations that might be due to the particular functional
form assumed in the analysis. We have done that, with the residuals for peak acceleration shown as an example in Figure 8. We see no persistent trends in the residuals, indicating that our equations provide a good approximation to the observations when averaged over all events and distances. As we will show later, this is not to say that the data from any particular event could not have systematic departures from the predicted motions (which are intended to represent median motions averaged over the population of earthquakes of a specified size and distance).

Results for Continuous Velocity Variation

Even though the classification system used in the previous section is an improvement over older classification schemes based on qualitative descriptions of site geology, it would be better still to compute the site effect as a continuous function of shear-wave velocity, if available. We have done that, generally following the ideas of Joyner and Fumal (1984). The term

\[ b_V (\log V_S - \log V_A) \]  

replaces the term

\[ b_S G_B + b_V G_C \]

in equation (1), where \( V_S \) is the time-averaged velocity at a site, and \( b_V \) and \( V_A \) are parameters determined from regression analysis. The values of the coefficients for different oscillator periods and dampings are given in Boore et al. (1994). The amplification given by equation (2) is shown in Figure 9 for a set of eight oscillator periods uniformly distributed logarithmically between 0.1 and 2 seconds. The plots show strong correlation of long-period ground motion with shear-wave velocity, consistent with the results when discrete classes are used to describe the geologic conditions at sites.

The dependence of the amplification on shear velocity is given by the coefficient \( b_V \) in equation (2); it was determined from earthquakes in California. As shown in Figure 10, the velocity dependence is remarkably similar to that determined by Midorikawa (written comm., 1993) in Japan and to the coefficients proposed by Borcherdt (1994) for use in determining short- and mid-period amplification factors in building codes.

Comparison of Results with Data from Specific Earthquakes

The peak acceleration predicted by the equations of Boore et al. (1993) are compared to the data from the 1989 Loma Prieta earthquake in Figure 11. The figure shows that there is a well-defined dependence of the peak accelerations on site class (this is the first earthquake clearly to show this effect). Also note that, on the average, the observed accelerations are consistently above the predicted values for each site class. This is not a troubling result, for the predictions are for the median value of a whole population of events with a magnitude equal to that of the Loma Prieta earthquake; we would expect some of the events to have values consistently above the predicted values and some to have values that are consistently lower than the predictions.
There is a suggestion that the distance-dependence of the data is not the monotonic decay of the predicted motions, although the scatter in the data makes it difficult to be certain of this point. In particular, as pointed out by P. Somerville and colleagues (e.g., Burger et al., 1987; Somerville and Yoshimura, 1990), the curvature of the rays traveling through the Earth’s crust and reflections from the layers within the Earth (particularly the Mohorovičić discontinuity) should produce a decay at short distance more rapid than inverse distance, followed by a flattening or an increase of amplitude in the 50 to 100 km distance range (depending on earthquake depth, crustal thickness, and velocity contrast at the discontinuity). The recordings of the mainshock may be too sparse along any given path to show clearly these effects, but this is not true of aftershock recordings made on portable instruments installed after the mainshock. Figure 12 shows the distance attenuation determined by Fletcher and Boatwright (1991) for a line extending from the source region to San Francisco. Their study shows the expected variations relative to simple inverse-distance geometrical spreading.

Comparable comparisons to the Loma Prieta data are shown in Figures 13 and 14 for recordings of the 1992 Landers main- and aftershocks, and similar remarks apply. As found by Mori (1993), the distance variation of the Landers aftershock data are strongly dependent on the path traversed from the source to the station. This dependence on path (presumably due to differences in crustal structure) may explain why the residuals of the data relative to the predicted attenuation from the analysis by Boore et al. (1993) of many earthquakes in western North America do not show the more complicated character expected for motions traversing a single path; these effects are averaged out. The Landers aftershock data also indicate that a substantial portion of the scatter in the strong-motion data about the mean predictions can be due to variations in crustal structure.

A comparison of the attenuation of motions from both the Loma Prieta and the Landers earthquakes is shown in Figure 15. An arbitrary vertical adjustment of the data was applied such that the two data sets approximately coincide at 40 km. If this is a valid normalization, the figure shows that the attenuation of motions along the San Francisco Peninsula from a source to the south is similar to that along a path from the Landers aftershock area to station GSC.

Subduction Earthquakes in the Pacific Northwest


In evaluating the published studies, it must be noted that some regression studies of data from Japanese earthquakes found the attenuation of motion with distance to be much less rapid than found for shallow earthquakes in western North America. Fukushima and Tanaka (1990), however, showed that these results were caused by use of unweighted least-squares regression. A properly weighted regression analysis applied to the same data found
that the distance attenuation was similar to that from shallow western North American earthquakes.

Earthquakes in Central and Eastern North America

Basis of Predictions

In many regions, such as central and eastern North America (which, for convenience, we refer to as “CENA”), too few strong-motion data are available for making empirical estimates. This is clearly shown in Figure 16, which plots a symbol for each recording as a function of magnitude and distance. Comparison of Figure 16 with the plot for western North America (Figure 4) shows that in CENA there are many fewer data in the critical range of earthquakes with magnitudes greater than 5, at distances less than 200 km. To emphasize the point, shaded lines have been added to Figure 16 to indicate the range of magnitudes and distances in Figure 4. Why not estimate ground motion theoretically? The theory of seismic wave propagation is sufficiently well developed so that ground motion could be calculated if all the necessary information were available. The catch is that the needed information is not available, and, in our view, probably will not be within the foreseeable future, if ever. The fault rupture that is the source of ground motion may be 10 km or more beneath the surface of the earth. It would be necessary to know the properties of all the material along the propagation path between the fault at depth and the site on the surface where the ground-motion estimate was wanted. The amount of slip on the fault would have to be known, and, more importantly, how that slip varied from point-to-point on the fault. A new approach has been developed recently, however, that makes theoretical estimates of earthquake ground motion possible. It combines simple approximations to wave-propagation effects with a statistical description of the variability of slip on the fault. Estimates made with this approach agree very well with the recorded ground-motion data that is available, and the approach makes possible what, in our view, are the first realistic theoretical estimates in regions for which there are few data.

The model is usually referred to as the stochastic model. The model was first proposed by Hanks and McGuire (1981) and was later developed by several authors (e.g., Boore, 1983; Boore and Joyner, 1984; Toro and McGuire, 1987). The essence of the stochastic model is shown in Figure 17. The energy radiated from an earthquake is assumed to be distributed randomly over a time interval determined by the source duration prolonged by arrivals from multiple travel paths. The idea that the motions are random has been used for years by engineers as a basis for their derivation of design motions; the crucial difference in the stochastic model is that the spectral content of the motions is given by seismological models, and therefore there is a physical and observational basis for the amplitude and relative frequency content of the motions. Extrapolations to situations lacking empirical data can be made with more confidence than is the case with previous models based on random processes. The model is very flexible and can be readily modified to incorporate various source-scaling relations, path effects, or site effects. It has the benefit that the computations are very rapid, and the sensitivity of motion to input parameters can be easily tested. The model is not limited in its application to CENA. The model has been verified
by comparison with world-wide data over a wide range of magnitudes and frequencies (e.g.,
Boore, 1983, 1986a,b). The model was first applied to CENA ground-motion estimation
by Atkinson (1984), who used it to predict peak acceleration and velocity on hard-rock
sites. Boore and Atkinson (1987) and Toro and McGuire (1987) extended the model to the
estimation of response spectra, and Boore and Joyner (1991) included the effect of deep soil
on ground motion. The stochastic-model-based attenuation equations published in 1987
by Boore and Atkinson and by Toro and McGuire are currently being revised by those
authors. Other applications include those of Chapman et al. (1990), Campbell (1991), and
Toro et al. (1992). In addition, many of the applications of the stochastic model to ENA
have been made by large projects directed by the Electric Power Research Institute and
by the Lawrence Livermore Laboratory; much of that work has not been published (but
see Toro et al., 1988, and Bernreuter et al., 1989).

In applications to CENA, there is a parameter with units of stress (usually symbolized
as Δσ; see Boore, 1983, for more explanation) that has a direct influence on ground motion,
particularly at the periods of most concern to engineers. The sensitivity of ground motions
to this parameter is shown in Figure 18. Unfortunately, Δσ is perhaps the least well
constrained of the important model parameters. Estimates of this parameter for CENA
earthquakes range from less than 50 to over 400 bars, with a median value near 150 bars
(Atkinson, 1993). This variation gives rise to considerable random variation of individual
ground motions about the median values.

There is some indication that the median value of Δσ is larger in CENA than in
western North America. Another model parameter that seems to be different in the two
regions is the parameter controlling the attenuation of seismic energy at high frequencies
(this is modeled in Figure 17 by a high-frequency corner beyond which the spectral
amplitudes decrease rapidly). The rock underlying sites in western North America is
commonly highly fractured and deeply weathered. In contrast, rock sites in eastern North
America (particularly those areas that were overridden by glaciers during the Pleistocene
epoch) are typically very hard, with little weathering and few fractures. This difference
shows up in CENA earthquakes at hard-rock sites having much more short-period motion
at distances close to the source than western North American earthquakes. An example
of this is shown in Figure 19. The acceleration spectra have been normalized to unity
at a period of 0.3 sec to account for the difference in magnitude of the events (4.0 for
Miramichi and 5.3 for Daly City); both recordings were obtained within 10 km of the
earthquake source. The Daly City acceleration spectra have peaks in the period range of
0.1 to 0.3 sec, but the peak for the Miramichi recording occurs at a period smaller than
0.05 sec.

Application to Rock Sites

Some of the results from an application of the stochastic model to prediction of ground
motions at hard-rock sites in CENA have already been presented in Figure 18. Further
results are contained in Figure 20, which shows the attenuation of various ground-motion
measures as a function of distance for a suite of magnitudes. Note that the separation
between the curves is greater for low-frequency than for high-frequency oscillators or for peak acceleration, just as it is for the empirical results for western North America (Figure 6). This difference in magnitude scaling is a direct consequence of the seismological source model shown in Figure 17: the difference of 2 units in moment magnitude (5 to 7) requires that the source spectra differ by 3 orders of magnitude at very low frequency and that the corner frequencies \( f_0 \) in the figure differ by a factor of 10. Purely geometric considerations then lead to a smaller difference between the curves at frequencies above the corner frequencies \( f_0 \).

### Application to Soil Sites

Not all of CENA is underlain by hard rock at the surface, and for those sites the lower-velocity near-surface sediments must be accounted for in the predictions of ground motion. This can be done using site-specific studies or by deriving correction factors for generic soil profiles. Figure 21 shows a compilation of shear-velocity profiles from a number of deep-soil sites, with an average profile indicated by the heavy line. Boore and Joyner (1991) derived correction factors for this profile and published equations from which deep-soil ground motions in CENA can be obtained. Figure 22 shows that correction factor (the factor that has to be added to the logarithm of the response spectral values on rock) for Boore and Joyner's generic soil, and for two site-specific soil profiles. The predictions of response spectra on soil and rock are shown in Figure 23.

### Comparison of Ground Motions in Western and Eastern North America

A comparison of ground motions on rock sites in western and CENA predicted from methods discussed in this paper is shown in Figure 24. Note the large difference in the response amplitudes at short periods, similar to that seen in the comparison of spectra from the Daly City and Miramichi recordings (Figure 19).

The difference in short-period content manifests itself in another way. Response spectra calculated using Newmark and Hall's scheme (Newmark and Hall, 1982) in which peak displacement, velocity, and acceleration are multiplied by amplification factors may not result in enough short-period response; an example of this is shown in Figure 25. The source of the problem is that the amplification factors are based on western North American earthquakes, but as we have shown, ground motions at close distances to earthquakes in CENA can be much richer in short-period energy.

### Acknowledgments

We thank Jack Boatwright, S. Midorikawa, and Jim Mori for data and curves, Walt Silva for the site-specific correction factors shown in Figure 22, and Linda Seekins for providing a thorough review. This work was partially funded by the U. S. Nuclear Regulatory Commission.
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Table 1. Definition of site class

<table>
<thead>
<tr>
<th>SITE CLASS</th>
<th>RANGE OF SHEAR VELOCITIES*</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>greater than 750 m/s</td>
</tr>
<tr>
<td>B</td>
<td>360 m/s to 750 m/s</td>
</tr>
<tr>
<td>C</td>
<td>180 m/s to 360 m/s</td>
</tr>
<tr>
<td>D</td>
<td>less than 180 m/s</td>
</tr>
</tbody>
</table>

* Shear velocity is time-averaged over the upper 30 m, computed by dividing 30 m by the travel time to 30 m.
Figure 2. M versus $m_{bLg}$, revised from Boore and Atkinson (1987). The curve labeled “AB87” is from Atkinson and Boore (1987). Note that although M is the more fundamental source parameter, the short-period magnitude $m_{bLg}$ is used for the abscissa because in most applications in central and eastern North America, the design earthquake will be in terms of $m_{bLg}$, and a moment magnitude M must be derived from this in order to make ground-motion estimates using seismological models.
Figure 1. The functional dependence of various magnitudes on moment magnitude. The relation for $m_{bLg}$ comes from Atkinson and Boore (1987). For $M_S$ and $M_L$ the relations came from fitting a quadratic to the data compiled by Ekström (1987) and Hanks and Boore (1984), respectively.
Distance Measures (from recording station)

D1 - Hypocentral
D2 - Epicentral
D3 - Closest distance to high-stress zone
D4 - Closest distance to fault rupture
D5 - Closest distance to surface projection of rupture

Figure 3. Diagram illustrating different distance measures used in relationships for estimating ground motion. (Modified from Shakal and Bernreuter, 1981.)
Figure 4. The distribution of the western North America data in magnitude and distance space (each point represents a recording). The data points labeled "old data" were used in previous studies (Joyner and Boore, 1981, 1982); the "new data" were added in the recent work of Boore et al. (1993). The points in the top and bottom frames were used in developing equations for peak acceleration and response spectra, respectively. (From Boore et al., 1993.)
Figure 5. Attenuation with distance of peak acceleration and response spectra for the random horizontal component. (from Boore et al., 1993).
Figure 6. Five-percent damped pseudoacceleration response spectra for a randomly-chosen component of horizontal motion at 10 km for site class B and a suite of magnitudes, using the equations of Boore et al. (1993).
Figure 7. Five-percent damped pseudoacceleration response spectra for a randomly-chosen component of horizontal motion at 10 km for magnitude 7.5 and a suite of site classes, using the equations of Boore et al. (1993).
Figure 8. Residuals of peak acceleration ($\log Y_{\text{observed}} - \log Y_{\text{predicted}}$), as a function of distance for magnitude groups and site classes. (From Boore et al., 1994.)
Figure 9. Amplification as a function of average shear velocity, as given by equation (2). The dots are the data used to determine the velocity dependence. (From Boore et al., 1994.)
Figure 10. The coefficient that controls the shear-velocity dependence of response spectral amplification, as determined by Boore et al. (1994) for California data and by Midorikawa (written communication, 1993) for data from Japan. Also shown are the coefficients proposed by Borcherdt (1994) for determining short-period and mid-period amplification factors in building codes; these were determined from Fourier amplitude spectra of recordings from the Loma Prieta earthquake.
Figure 11. Peak horizontal acceleration as a function of distance for the 1989 Loma Prieta earthquake, compared with predictions from Boore et al. (1993).
Figure 12. Decay of Fourier spectral amplitudes for aftershocks recorded along a line from the Loma Prieta source region to San Francisco (Fletcher and Boatwright, 1991), compared to the attenuation of response spectra determined by Boore et al. (1993) at a period consistent with the frequencies of the Fourier spectral components. The relative placement of the response spectral and Fourier amplitude data on the ordinate is arbitrary; only the relative change in amplitude with distance is important. We have normalized the curves to unity at 10 km.
Landers (6/28/92)

Figure 13. Peak horizontal acceleration as a function of distance for the 1992 Landers earthquake, compared with predictions from Boore et al. (1993).
Figure 14. Decay of amplitudes for Landers aftershocks recorded at GSC and at PFO (from Mori, 1993), compared to the attenuation of response spectra determined by Boore et al. (1993) at periods consistent with the frequencies of the aftershock recordings. The aftershocks of the Landers earthquake are distributed along a more-or-less linear trend extending for more than 80 km, with stations GSC and PFO to the north and south of the aftershock zone, respectively. Mori (1993) took advantage of the source and station distributions to construct profiles in which the source location changed and the recorder location was fixed, the inverse of the usual situation. This eliminates variations in site response, but requires a normalization for the varying magnitudes of the aftershocks. Mori (1993) normalized the motions to a common magnitude. The relative placement of the response-spectral predictions and the aftershock data on the ordinate is arbitrary; only the relative change in amplitude with distance is important. We have normalized the curves and an approximate average of the aftershock data to unity at 40 km. Note the very different amplitude dependence for paths to GSC and to PFO (the azimuths of the paths from the source regions to these stations differ by approximately 180 degrees).
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Figure 16. Distribution in magnitude and distance of ground-motion recordings in central and eastern North America. Three symbols have been used to indicate whether the recording was from an accelerograph (primarily SMA-1 analog recorders), a standard seismological station, whose output is proportional to velocity for most frequencies (most of these data are from the ECTN, as provided by G. Atkinson), or a blast recorder (Street et al., 1987; Street et al., 1988). The shaded line outlines the boundaries of the plot showing the magnitude and distance distribution for recordings in western North America (Figure 4).
Figure 17. Basis of stochastic model. Radiated energy described by spectra in the upper part of the figure is assumed to be distributed randomly over a duration equal to the inverse of the lower-frequency corner \( f_0 \). The time series in the lower part of the figure are one realization of a random process. The levels of the low-frequency part of the spectra are directly proportional to seismic moment. The curves and time series shown are from an actual simulation and indicate the difference in amplitude and duration expected for these two magnitudes. (From Boore, 1989.)
Figure 18. Ground motions for a range of stress parameters. (From Boore and Atkinson, 1987.)
Figure 19. Acceleration response spectra, normalized to the value at 0.3 sec, for rock recordings of the 1957 Daly City earthquake in California and an aftershock of the 1982 Miramichi, New Brunswick, earthquake.
Figure 20. Ground motion at hard-rock sites as a function of distance for a suite of moment magnitudes. (From Boore and Atkinson, 1987.)
Figure 21. Shear velocity as a function of depth for an average deep-soil site (heavy line), based on a compilation of velocities from PSAR and FSAR reports for nuclear power-plant sites throughout the United States (light lines). The short horizontal bars indicate bedrock. (From Boore and Joyner, 1991, who adapted it from a figure in Bernreuter et al., 1989, who in turn obtained it from W. Silva (personal communication).)
Figure 22. Site coefficients ("e" in equation (1)) as a function of oscillator period for a generic firm, deep-soil site (from Boore and Joyner, 1991) and two specific sites in the vicinity of New Madrid, Tennessee (see Toro et al., 1992 for a discussion of the soil profiles; the figure was constructed from data provided by W. Silva).
Figure 23. Comparison of pseudoacceleration spectra in central and eastern North America at hard-rock and deep-soil sites, both based on the same methodology. The comparison is for moment magnitudes of 6.5 and 7.5 and hypocentral distances of 20 and 100 km. (From Boore and Joyner, 1991.)
Figure 24. Comparison of acceleration response spectra at rock sites in western North America (WNA) and eastern North America (ENA). The WNA results are from the empirical analysis of Boore et al. (1993), and the ENA results are computed using the stochastic model and the parameters of Boore and Atkinson (1987) and Boore and Joyner (1991). The WNA results are given for site classes A and B. Note the large difference in spectral acceleration at short periods, similar to the results in Figure 19.
Figure 25. Pseudovelocity response spectra at 20 km as a function of oscillator period for a hard-rock site in central or eastern North America. The heavy lines give the predicted values from the stochastic model. The light lines were obtained using Newmark and Hall's median amplification factors (Newmark and Hall, 1982, Table 1), with faring to the short-period asymptote beginning at 0.125 sec and ending at 0.03 sec.
Abstract

The USGS has published national ground-motion hazard maps since 1976. These national maps have been used in constructing design value maps for building codes. The ground-motion maps are based on a probabilistic methodology which considers the rate of earthquake occurrence in area source zones and along major faults. The probabilistic method can be applied to assess seismic hazard on a consistent national basis. The most recent USGS national maps show spectral response values at 0.3 and 1.0 sec periods with a 10% probability of exceedance in 50 and 250 years. A method for using these two spectral values to construct complete response spectra is described. Uniform hazard spectra have differing shapes for cities in the western and eastern U.S., because of differences in recurrence times of large events and differences in attenuation of seismic waves. Deterministic methods of ground-motion prediction can be used when scenario earthquakes can be specified. We describe some of the relevant issues in making hazard maps for southern California, the San Francisco Bay region, the Pacific Northwest, the central U.S., and the northeastern U.S. Our present and future efforts consist of producing regional hazard maps incorporating detailed geologic mapping of site conditions. In addition, we are in the process of re-assessing seismic source zones, in light of recent research, to produce the next-generation of national hazard maps.

Introduction

There are two principal approaches to mapping ground-motion hazards from earthquakes: probabilistic and deterministic. Probabilistic hazard maps answer the question "what is the probability of exceeding a specified ground motion over a particular time period" or "what is the ground motion that has a specified probability of being exceeded over a particular time period?" The probabilistic method quantifies how often a location will experience ground motions of various levels (Cornell, 1968). The method requires estimates of the recurrence rates of earthquakes. This probabilistic representation of hazard has a direct application in the design of buildings. One approach to making a design value map is to use ground motions with an equal chance of being exceeded over the lifetime of a structure, say 50 years.

In the deterministic procedure, the location and magnitude of a scenario earthquake is specified and the resulting ground motions from that event are mapped.
deterministic approach answers the question "what will the ground motions be if the scenario earthquake occurs?" There is no consideration of exposure time or recurrence rates of earthquakes. This deterministic method can be used in regions where major faults have been identified and the likely magnitudes of future earthquakes have been determined. For example, we can apply the deterministic method to quantify ground motions for future large earthquakes on the San Andreas Fault in California. In the eastern U.S., however, it is usually impossible to identify specific faults where future large earthquakes will occur. Deterministic maps of hazard in the east are generally restricted to assessing the ground motions for future earthquakes located near the epicenters of large earthquakes that have occurred in the past.

The deterministic approach has been used to make a national hazard map based on the observed intensities of past earthquakes (Algermissen, 1969). This map assigns high hazard to areas which have experienced large intensities from past earthquakes. It does not consider the recurrence rate of future ground shaking for each area, nor does it take into account the possibility that future large earthquakes may occur in regions that have not had large events in the recent past.

Since 1976, the USGS has used the probabilistic method to quantify the seismic hazard on a uniform national basis. The first USGS probabilistic map for the U.S. was published by Algermissen and Perkins in 1976. Subsequent national maps were published as Algermissen et al. (1982) and Algermissen et al. (1990). These maps showed peak horizontal ground acceleration (PGA) with a 10% chance of being exceeded in 50 and 250 years. These maps are based on a soft-rock site condition.

The latest USGS national maps were published in Algermissen et al. (1991) and Algermissen and Leyendecker (1992) and depict pseudo-spectral acceleration (PSA or $S_A$) response values at 0.3 and 1.0 sec periods (5% damping) with a 10% chance of exceedance in 50 and 250 years. The 50 year exposure maps are shown in Figure 1. The mapped values are given in percentage of the acceleration of gravity g (note, PSA at 0.3 sec period is approximately equal to the peak ground acceleration multiplied by 2.5). These recent maps are for an S2 soil condition. All of the USGS national probabilistic maps are based on a time-independent (Poisson) model of earthquake occurrence, that is, the occurrence of any one earthquake is not related to the occurrence of previous earthquakes.

The USGS probabilistic maps have been the starting point for design value maps used in many building codes. The seismic zonation map used in the Uniform Building Code (UBC) is largely based on the 1976 maps (10% chance of exceedance in 50 years), with truncation of probabilistic ground motions greater than 0.4g PGA and the placement of floors of ground motions in some areas, including Oregon and the central valley of California. The 1976 maps also form the basis of the design value maps in the 1985, 1988, and 1991 versions of the NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, produced by the Building
Seismic Safety Council (BSSC). These design value maps truncate ground motions greater than 0.4g PGA, but unlike the UBC, do not place a floor on ground motions in the central valley of California. The Appendix to the 1991 NEHRP Recommended Provisions contains the latest USGS spectral response maps and specifies that these maps are to be evaluated for use as possible design value maps.

We emphasize that probabilistic ground-motion maps are not necessarily equivalent to design value maps. Past practice has included substantial engineering judgement and other factors in producing a design value map from a scientifically-derived map of probabilistic ground motions.

This paper has two principal purposes. First, we describe the probabilistic methodology used in the hazard maps. Second, we detail some of the issues involved in producing hazard maps for the regions of the U.S. where the ATC-35 seminars will be given.

Probabilistic Methodology

The first step in a probabilistic analysis is to characterize the spatial and temporal nature of seismicity in the region of interest. Two basic types of sources are considered: line sources (faults) and areal source zones with uniformly distributed seismicity. Figure 2 depicts schematically a site where the total seismic hazard is calculated by summing the hazard from various types of source zones. This figure also shows the essential information needed to construct a probabilistic hazard map: (1) frequency of earthquake occurrence as a function of magnitude for each source zone and (2) ground motions as a function of magnitude and distance.

Figure 3 contains the source zones for the U.S. used in Algermissen et al. (1982, 1990, 1991), along with historical seismicity. The source zones were chosen to outline areas of clustered seismicity and/or to encompass geologic structures which are thought to have seismogenic potential. A series of workshops were held in 1979-1980 to elicit the input of regional experts on the boundaries of these source zones. We will be conducting another series of regional workshops of earth science experts in 1994 and 1995 to determine source zones for the next generation of national hazard maps. The new maps will be submitted for consideration of BSSC for the 1997 NEHRP Recommended Provisions. They will also be available for consideration by the model codes and national standards.

For each source zone, the annual rate of occurrence is specified as a function of magnitude. This can be found in terms of the $a$ and $b$-values, which are the $y$-intercept and slope, respectively, on a plot of the logarithm of the number of earthquakes (or frequency of occurrence) as a function of their magnitude (see Figure 2b). This frequency-magnitude plot is derived from catalogs of historical and instrumental earthquakes in that source zone, after removal of aftershock sequences. The $a$-value
characterizes the overall rate of earthquakes, while the $b$-value specifies the proportion of small earthquakes to large ones. In some cases, there is insufficient seismicity in a given source zone to produce reliable estimates of the $a$ and $b$-values. In these cases, the $a$ and $b$-values are determined by combining the seismicity in several source zones with similar tectonic settings. The combined rate of earthquakes is then apportioned into the source zones based on their relative rates of activity. It is also necessary to choose the maximum magnitude earthquake that can occur in each source zone. Needless to say, this is an uncertain parameter, especially for the central and eastern U.S.

We illustrate the probabilistic method by describing how to calculate the probability of exceeding some specified ground motion at a site due to earthquakes in a single source zone. First we consider the probability of exceeding ground motion $u_0$ at that site from earthquakes occurring in a unit area within the source zone (Figure 4). Let's initially consider the ground motions from earthquakes with magnitudes between 6 and 7. Now the annual probability of exceeding $u_0$ is

$$P(u > u_0) = \frac{\lambda}{A} P(u > u_0 | \text{EQ})$$

Here $\lambda$ is the annual rate of magnitude 6 earthquakes in the entire source zone and $A$ is the area of the source zone.

$P(u > u_0 | \text{EQ})$ is the probability that ground motion will exceed $u_0$ at that site, given that an earthquake with the specified magnitude occurs at the designated location in the source zone. This term is derived from the attenuation relations which give the ground motion amplitude as a function of distance and magnitude (Figure 2c), as determined from observed strong motion data. This term is also a function of the observed variability of ground motions which is discussed in detail in the next paragraph.

For any given magnitude and distance, earthquake ground motions will vary about some average value, due to differences in site conditions and source properties, such as stress drop, directivity of rupture, and the location of strong patches (asperities) on the fault. Figure 5 shows an idealized probability density distribution of ground motions about some average value, for a particular magnitude and distance. This distribution is log-normal. Thus we are equally likely to observe ground motions $1/2$ the mean and twice the mean, for example. The mean here is calculated from averaging the logarithms of the ground motions. This variability is explicitly quantified in regressions of strong motion data. Furthermore, this variability is not caused by uncertainties in measurements, but rather by variations in physical parameters associated with the source, site and path. The probability of exceeding $u_0$ is the area under the probability density function for ground motions greater than $u_0$. It is this probability that is the right hand factor in equation (1). In this way the probabilistic method incorporates the variability of ground motions.
Returning to equation (1), we next have to sum the probability from the right hand side over all possible unit areas within the source zone and over all relevant magnitude ranges. Typically, a magnitude of about 4.5 to 5 is used as a minimum magnitude in the probabilistic calculations. Then the annual probability for exceeding that ground motion has to be added together for all the source zones.

In order to calculate equation (1), we need a relationship between ground motion and magnitude and distance. For the latest spectral maps, the attenuation relations from Joyner and Boore (1981) were used for the western U.S. These are based on strong-motion data. These empirical attenuation relations also find the standard deviation of ground motions from the observed variability of ground motions. For the central and eastern U.S., the attenuation curves from Boore and Joyner (1991) were used. These curves were derived from stochastic simulations of seismograms using path and source parameters found from the analysis of earthquake recordings in the central and eastern U.S.

Once the total annual probability of exceedance \( P(u > u_0) \) for all magnitudes and source zones is calculated for a site, the Poisson probability \( P(u > u_0 | t) \) for a given exposure time \( t \) can be calculated by

\[
P(u > u_0 | t) = 1 - e^{-P(u > u_0)t}
\]

For small probabilities, the Poisson probability for \( t \) years is approximately the annual probability times \( t \). Thus, a 10% probability of exceedance in 50 years is about equivalent to an annual probability of exceedance of 0.1/50 or 0.002. This case corresponds to ground motions with an average repeat time of 500 years. Similarly, ground motions with a 10% chance of exceedance in 250 years have an average repeat time of about 2500 years.

A seismic hazard curve is calculated for each site with the probability of exceedance derived from equation (1) plotted as a function of ground motion level \( u_0 \) (Figure 6). This hazard curve is then used to find the ground motion with the desired probability of exceedance. This is the value assigned to that location on the hazard map. After the probabilistic ground motions for a fixed probability are found for each grid location, a contour or color map of the probabilistic hazard is produced.

The latest USGS maps show spectral response values at periods of 0.3 and 1.0 sec with a 10% probability of exceedance in 50 and 250 years. These two periods were chosen so that the shape of the response spectrum could be specified for design purposes. Most previous design methods used a standard shape of the response spectrum with its absolute level fixed by the peak acceleration. The problem with this approach is that response spectra derived from the probabilistic method do not have the same shape for all regions of the country.

The probabilistic method can be used to construct response spectra where the response spectral values at all periods have the same probability of exceedance. Such
response spectra are called uniform hazard spectra (UHS). Examples of UHS for
different cities of the U.S. are shown in Figure 7 (10% chance of exceedance in 50
years). These spectra show the relative hazard for each city at each period. In Figure 8,
these spectra have been normalized to have the same value at 0.3 sec. This normalization
was done so that the relative amounts of long-period and short-period amplitudes
could be compared.

The shapes of the UHS can be roughly divided into two classes. The normalized
UHS for California cities (San Francisco, Los Angeles, Oakland, and San Diego) have
much higher amplitude at periods of 1.0 sec and longer than do the normalized UHS
for Salt Lake City and cities in the eastern and central U.S. (Memphis, New York, St.
Louis, Chicago, and Charleston). In other words, the UHS for the California cities are
enriched in long-period energy relative to the UHS in the other cities. This is largely
caused by the difference in the repeat times of large earthquakes between the regions.
The repeat times for large ($M \geq 7$) earthquakes in California is on average much
shorter than those in the Salt Lake City region and the central and eastern U.S.
Therefore, the large earthquakes will contribute more to the probabilistic hazard for an
exposure time of 50 years in California than the other regions. Large earthquakes generate
more long-period energy than short-period energy, compared to smaller earthquakes.
This causes the UHS in California cities to be enriched in the longer period energy. A
secondary reason for the difference in spectral shapes is the lower attenuation in the
central and eastern U.S., compared to California.

By specifying response spectral values at 0.3 and 1.0 sec, one can approximate
the shape of the UHS for periods ($T$) from 0.1 to 4.0 sec (see Figure 9). Algermissen
and Leyendecker (1992), Leyendecker and Algermissen (1994), and Leyendecker et al.
(1994) approximate the acceleration response spectrum at short-periods with a flat level
equal to the 0.3 sec value. At longer periods the response spectra is approximated by a
$1/T$ falloff whose amplitude is fixed by the 1.0 sec value. Approximating the UHS in
this manner does a good job in enveloping the complete response spectra for California
cities (Figure 9). This method underestimates the response spectra for periods less than
about 0.2 sec for the cities in the eastern and central U.S. A $T^{-2/3}$ falloff to the
response spectrum at periods greater than 1 sec has been proposed for the design spec-
trum in building codes (Figure 9), to introduce conservatism at long periods.

Regional Issues in Seismic Hazard Assessment

In the next portion of this paper, we discuss some of the relevant issues for
seismic hazard assessment in the regions where the ATC-35 seminars will be given.
This discussion is not intended to be complete. Our intention here is to highlight cer-
tain issues in each region to give the reader a sense of the difficult problems involved
in quantifying seismic hazard. The USGS is actively engaged in addressing these
problems in each region, to produce improved national and regional maps of seismic hazard.

**Pacific Northwest**

Figure 10 (left) shows the 0.3 sec PSA for a 10% chance of exceedance in 50 years for the Pacific Northwest, excerpted from the national map of Algermissen et al. (1991). This map was constructed by considering the three classes of seismic sources for the Pacific Northwest: 1) large (moment magnitude $M \geq 8$) thrust earthquakes on the Cascadia subduction zone offshore of Oregon and Washington, 2) earthquakes at depths of 50-80 km on the subducted plate, and 3) earthquakes within the crust. Although no great earthquakes have occurred historically in the past two hundred years in this area, there is abundant evidence from drowned coastal areas of the past occurrence of such events (Atwater, 1987). In addition, there is recently-discovered evidence that the last event about 300 years ago produced paleoliquefaction features found on islands in the Columbia River (Obermeier et al., 1993).

It is problematic how to incorporate the subduction zone earthquakes into the probabilistic hazard maps (Perkins and Hanson, 1993). The probabilistic maps for a 50 year exposure time are only weakly influenced by the ground motions from earthquakes with repeat times of 500 years or more. For the map shown in Figure 10 (left), it was assumed that one magnitude 8.25 event occurred every 500 years on the subduction zone. Evidence from drowned coastlines, however, indicate two possible scenarios: 1) a magnitude 9 earthquake that ruptures the entire subduction zone about every 500 years or 2) a series of magnitude 8 events that occur closely-spaced in time that fill the entire rupture zone about every 500 years.

Figure 10 (right) shows the probabilistic hazard map for the case of M8.25 events filling the subduction zone every 500 years (average recurrence time of 100 years). The biggest changes from the map to the left are the larger values near the coast. Contours further inland now are parallel to the coast and values are somewhat larger than for the case on the left. The problem with the scenario on the right is that it predicts one M8.25 earthquake every 100 years and none has been observed historically over the past 200 years. Preliminary work indicates that a scenario with one M9 earthquake every 500 years produces probabilistic ground motions (10% chance of exceedance in 50 years) somewhat less than those for five M8.25 earthquakes every 500 years (Perkins and Hanson, 1993).

Some seismologists and engineers in the Pacific Northwest think it is prudent to design buildings to withstand a great subduction zone earthquake. This would be a deterministic approach. One of their arguments is that, given the uncertainty of our knowledge about the subduction zone earthquakes, we should design for a worst-case scenario. One could extend this reasoning to the fault discovered by Bucknam et al. (1992) that passes through southern Puget Sound and has produced a large earthquake...
within the past 1700 years. Its repeat time is not well constrained. Should we design structures to withstand a large earthquake on this fault adjacent to downtown Seattle? These are issues that go beyond a scientifically-based ground motion map into the realms of public policy and economics. As seismologists, we can provide probabilistic and deterministic maps of hazard and their uncertainties, but we cannot make judgments on what is an acceptable level of risk to a community.

Another problem in the Pacific Northwest is how to assess the hazard from potential earthquakes at 50-80 km depth on the subducted plate. Large damaging earthquakes have occurred on the subducted plate in the Puget Sound region in 1949 (M7.1) and 1965 (M6.5). However, no large (M ≥ 6) earthquakes have been observed historically or instrumentally on the subducted slab beneath southern Washington and Oregon. The USGS maps are based largely on historical seismicity and, therefore, assumes that large earthquakes will be less likely on the subducted plate under Portland than under the Puget Sound area. Again, some would argue that we should design structures to withstand shaking from a large, intermediate-depth earthquake under Portland, regardless of its likelihood based on the historic record.

The Klamath Falls, Oregon earthquakes (M6) of 1993 point out the hazard from crustal earthquakes along faults in the Cascade Range. Geologists are just beginning to quantify slip rates for Quaternary faults along the Cascade Range in Oregon and Washington (Pezzopane and Weldon, 1993). As we re-assess the source zones in the national maps, we will incorporate the latest findings on these faults.

Regional hazard maps for urban areas of the Pacific Northwest should include site response. There will be substantial amplification of seismic waves by the Puget basin and the Portland Basin. Mabey et al. (1993) have done extensive shear-wave velocity logging of the Portland area and have produced a map of the expected amplification of vertically-propagating shear waves. There is also likely to be 3-D basin effects in both regions. These effects can caused prolonged seismic shaking at periods of 0.5 sec and longer (see Frankel paper in this volume on basin response).

Central U.S.

Figure 11 shows the probabilistic ground motions for the central and eastern U.S. derived by Algermissen et al. (1991). The major zone of seismic hazard in the central U.S. is the New Madrid seismic zone which generated four earthquakes with moment magnitudes estimated at about 8 during 1811 and 1812. This area continues to have the highest seismicity rate in the central and eastern U.S.. Results based on extrapolating rates of seismicity of smaller events indicate an average repeat time of from 500-1100 years for a magnitude 8 earthquake in this zone (Johnston and Nava, 1985). Recent work on paleoearthquakes indicates repeat times for M8 earthquakes to be as short as 1000 years (Schweig et al., 1993). Probably the dominant source of hazard in this region over a period of 50 years is from a M6 earthquake (Johnston and Nava,
Two M6 earthquakes occurred in the region in 1843 and 1895. Calculating a recurrence rate based on only these two events is problematic. What does one choose for the beginning and ending points of the time window? It could also be argued that these two events are late aftershocks of the 1811-1812 earthquakes, so that extrapolating their rate of occurrence to the future may overestimate the rate. On the other hand, a recurrence rate of about one M6 event every hundred years is consistent with extrapolating the observed rate of smaller events.

Another relevant question is whether the M8 earthquakes represent characteristic earthquakes on the New Madrid Seismic Zone. Can this area generate earthquakes with magnitudes between 6 and 8? Should we assume an exponential decay in the number of events as a function of magnitude between 6 and 8?

Differences in assumptions can lead to large differences in seismic hazard maps for this area. Figure 12 compares the hazard maps for the New Madrid region developed by Toro et al. (1992) and Algermissen et al. (1990). Each map shows peak accelerations with a 10% chance of exceedance in 50 years. Note the differences between the two maps. The Toro et al. (1992) map considered regional differences in site response and used a non-linear soil response based on laboratory measurements of soil properties. The Algermissen et al. (1990) map used a uniform, soft-rock site condition. The maps differ by almost a factor of four at Memphis, with the Algermissen et al. (1990) map indicating substantial hazard to Memphis over 50 years (0.2g PGA), while the Toro et al. (1992) map showing relatively small hazard (0.05g PGA) to Memphis for the 50 year exposure time. Toro et al. (1992) suggest that most of the difference is due to the different rates of earthquakes used in the two studies. In our initial examination of this problem, it appears that the different rates are due to different assumptions about characteristic earthquakes in the New Madrid zones. Clearly, we need to do more work on hazard maps for this area, to identify and quantify the sources of uncertainty. This is the subject of an active, ongoing project in the USGS.

For the Memphis area, a key question is the seismic activity rate of the eastern boundary fault of the Reelfoot rift. This structure lies adjacent to Memphis and has significant microseismicity (M ≤ 3) associated with it. We do not know whether this structure is capable of generating a large (M ≥ 6) earthquake. The Toro et al. (1992) study assigned seismicity near this boundary fault into a broad source zone, which essentially diluted the hazard from this fault. We feel that a more localized source zone along the east rift fault is probably more appropriate for characterizing the hazard to Memphis.

The Wabash Valley seismic zone is the second area in the central United States where there is evidence that major (M7) earthquakes occurred in the past. This evidence consists of widespread paleoliquefaction features associated with four strong earthquakes that occurred in this area over the past 14,000 years (Obermeier et al., 1985).
1992; C.A. and P.J. Munson written comm., 1993). We are in the process of re-
assessing this source zone in light of these new findings. It is crucial to quantify the
recurrence rates of large earthquakes in this source zone. If the recurrence rate is
greater than a thousand years, such events would not be strongly represented on a pro-
babilistic ground motion map with a 10% chance of exceedance in 50 years. Again,
this is a situation where deterministic maps are useful supplements to probabilistic
maps.

Regional seismic hazard maps for the central U.S. should, in the future, include
the effects of amplification by surficial materials. The Mississippi Embayment strongly
amplifies seismic shear waves at frequencies less than 10 Hz. In addition, resonances
in the sedimentary column can occur at certain frequencies determined by the thickness
and shear-wave velocity of the sediments, properties that will change with location.
Harris et al. (1994) used seismic refraction profiles to map the resonant period and
amplification of sediments in the Paducah, Kentucky area. Eggert et al. (1993) meas-
ured shear-wave velocity in boreholes in the Evansville, Indiana area and mapped
resonant periods of soils. We need to collect more information on shear-wave velocity
of surficial materials as well as depth-to-bedrock information. Amplification from sedi-
ments is obviously important for cities in the region located in river valleys. In addi-
tion, future maps should consider the hazard from short-period (0.5-5 sec period) sur-
face waves. Such surface waves have been observed for paths crossing the Mississippi
Embayment and can have very long duration (see Frankel paper in this volume on
basin response).

Northeastern U.S.

Because of its high population density, seismic risk is high in the northeastern
U.S. despite a relatively moderate level of earthquake activity. The source zones
developed for this region largely reflect the spatial distribution of historical seismicity
(see Figure 3). One source zone encompasses most of New Jersey, eastern Pennsyl-
vania, southern New York (including New York City), and most of Connecticut. This
region has a history of small and moderate-size earthquakes up to about magnitude 5.
The seismicity seems to be diffusely distributed and cannot be associated with particu-
lar tectonic structures (Ebel and Kafka, 1991). A second source zone includes the Bos-
ton and Cape Ann areas of Massachusetts. This zone contains the Cape Ann earth-
quake of 1755 (M6). Other major source zones in this region are in the Adirondack
Mountains of upstate New York; the Lake Champlain-Vermont area; western New
York state near Attica; and the Appalachian trend in Maine and New Hampshire. Each
of these source zones has a history of small and moderate-sized earthquakes. The prob-
babilistic ground motions for this area are shown in Figure 11.

For all of these source zones, one problem is choosing the maximum magnitude
earthquake for the hazard assessment. Should we assume that each source zone is
capable of generating a moment magnitude 7.5 earthquake similar to the Charleston, South Carolina earthquake of 1886? Or, should we use the maximum earthquake observed historically in the northeast, which is about magnitude 6. In the present maps, a maximum magnitude was assumed to be 7.3 for the source zone containing New Jersey, southern New York, and Connecticut, since this region is characterized by the same geologic episodes of deformation and, therefore, similar geologic structures as the Charleston, South Carolina area. Most of the other source zones in the northeast were assigned a maximum magnitude of 6.7.

The probabilistic seismic hazard in the northeast in a 50 year exposure time is largely produced by the occurrence of earthquakes between magnitudes 5 and 6. Most events of this size in the historic catalog occurred before seismometers were deployed in this region. The recurrence rates assigned to each source zone are largely constrained by the historical intensity data. Jacob et al. (1994) determined the recurrence rates of moderate-sized events by extrapolating from the rates of magnitude 2 earthquakes recorded by a seismic network in the area. The probabilistic hazard map that they developed from this procedure had similar values as the Algermissen et al. (1990) maps in the New York City region, but significantly higher values than the Algermissen et al. map in the Adirondack area. However, more work needs to be done to assess the spatial correlation between areas with currently high levels of magnitude 2 events and areas which generate M5-6 events relatively frequently.

Southern California

The hazard in southern California region is due to large (M \geq 6.5) earthquakes on the major strike-slip (e.g. San Andreas) and thrust (e.g. Sierra Madre) fault systems in the region, as well as due to M5-6 earthquakes that can occur on smaller faults distributed throughout the region. Figure 13 contains the source zones used by Algermissen et al. (1982, 1990, 1991) for California (see Thenhaus et al., 1980). The major strike-slip fault systems (e.g., San Andreas, San Jacinto, Elsinore) are specified by elongated source zones. Areas of thrust tectonics (e.g. Santa Monica-Raymond Hill; Sierra Madre; Santa Barbara; Ventura Basin) are divided into several areal source zones. The activity rates for each source zone were determined from historical and instrumentally-located seismicity.

The probabilistic ground motions are highest along the San Andreas, San Jacinto, and Imperial Faults (Figure 14). This reflects their high rates of historical activity. For 10% probability of exceedance in 50 years, the probabilistic ground motions exceed 200\%g PSA at 0.3 sec along much of these fault zones. Another area of relatively high probabilistic ground motions is between Santa Barbara and Ventura. This is caused by the high rate of events in the source zone south of Santa Barbara. A ridge of high ground motions occurs along the Newport-Inglewood Fault southwest of downtown Los Angeles and extends southward along the coast to San Diego.
A major unresolved issue is how to treat the possible occurrence of major (M7) earthquakes on blind thrusts under the Los Angeles basin. There is clearly much deformation occurring in the basin along buried thrust ramps. However, only small and moderate-sized (M ≤ 6) earthquakes have been observed instrumentally in the Los Angeles area near these buried thrusts. The essential question is "how much of the deformation is released in large earthquakes?" This is a subject of active research. Until we understand more about this process, we should probably produce hazard maps for Los Angeles which consider a range of plausible rates of large earthquakes on the buried thrusts.

We are presently incorporating geologic information in hazard maps of California. This information can supplement the historical rates of large earthquakes, for cases where faults are aseismic between characteristic large earthquakes. Geologists have identified segments on the major faults which tend to rupture in earthquakes of a characteristic magnitude related to the segment length. Of course, some earthquakes rupture multiple segments. Assuming the characteristic rupture, the average recurrence time can be calculated from the geologic slip rate and the segment length. These geologically-derived recurrence rates can be used in the probabilistic hazard maps (Wesnousky, 1986; Petersen, 1992). Presently, the USGS and the Southern California Earthquake Center are both developing hazard maps of southern California using geologically-derived repeat times on the major faults as well as using areal source zones to assess the hazard from smaller events (M4.5-6.5).

We are focusing our initial efforts on producing ground motion hazard maps for the San Bernardino Valley (Frankel et al., 1994), located about 60 miles east of Los Angeles. These maps incorporate site amplification. The procedures used to make these maps can be followed as a template for making hazard maps in other regions. The surficial materials are classified according to their inferred shear-wave velocity (\(V_s\)) in the top 30m. Figure 15 shows our classification of surficial geologic materials for the San Bernardino Valley, based on the geologic map of California Division of Mines and Geology (1967). This classification scheme corresponds to that of Boore et al. (1993): class A with \(V_s > 750\) m/sec; class B with \(360-750\) m/sec; class C with \(V_s 180-360\) m/sec. As a crude initial step we assigned the bedrock to class A, Pleistocene alluvium to class B, and recent alluvium to class C. This assignment was based roughly on the work of Fumal and Tinsley (1985). We used the recent empirical attenuation relations of Boore et al. (1993), which give ground motion amplification factors for these three site classes. These relations were derived from observed strong motion data.

We have produced preliminary probabilistic and deterministic maps of hazard for the San Bernardino Valley, for earthquakes on the San Andreas and San Jacinto faults. Recurrence rates for the various segments of these faults were taken from the compilation of the Southern California Earthquake Center. This initial analysis does not
include the hazard from smaller earthquakes (M < 6.5) in the region. The probabilistic map (Figure 16) shows ground motions (0.3 sec PSA) with a 10% chance of exceedance in 50 years. For the deterministic map, we calculated the ground motions from characteristic earthquakes on each fault segment. We then plotted the largest ground motions found at each site. We plotted the mean plus one standard deviation ground motions in the deterministic map in Figure 17.

Both the probabilistic and deterministic maps largely reflect the spatial distribution of site conditions. The largest motions occur on the recent alluvium, the smallest motions occur on bedrock outside the San Bernardino Valley. The probabilistic ground motions are similar, in most cases, to the mean plus one standard deviation deterministic motions. In both maps, the ground motions in the Valley are generally between 200 and 250%g for 0.3 sec PSA. The probabilistic ground motions are highest in the northwest corner of the map, where the San Jacinto and San Andreas faults nearly converge. This is caused by the additive hazard from both faults (see Discussion section). The deterministic motions are higher near the San Andreas than the San Jacinto fault, because the characteristic magnitude is larger on this segment of the San Andreas (M7.2) than on this segment of the San Jacinto Fault (M6.9).

The probabilistic ground motions are about 250%g PSA (0.3 sec) for downtown San Bernardino (10% chance of exceedance in 50 years). We can also express the probabilistic hazard in terms of the probability of exceeding 100%g PSA at 0.3 sec. This is the nominal design value in the building code. Downtown San Bernardino has about a 57% probability of exceeding 100%g PSA (0.3 sec) in 50 years.

In practice these types of hazard maps are limited in their detail by the resolution of the geologic mapping and the uncertainty in shear-wave velocity of the surficial units. We are presently measuring shear-wave velocity of representative units in boreholes in the San Bernardino Valley to improve our hazard maps. We are also investigating 3-D basin effects at longer periods (> 0.5 sec) in this valley using computer simulations (see Frankel paper in this volume). As these modeling efforts are refined, they will be incorporated into the hazard maps.

San Francisco Bay Region

The seismic hazard maps in the San Francisco Bay region are dominated by the major strike-slip faults, the San Andreas and Hayward faults. The highest ground motions in the Algermissen et al. (1991) maps are along the San Andreas fault (see Figure 14).

We are presently re-evaluating the hazard for the San Francisco Bay region by incorporating geologic slip rates and site amplification (Thenhaus et al., 1993). We have compiled slip rates and segmentation data for the major faults in the region (Figure 18). From these data, we have calculated characteristic magnitudes and average recurrence rates of large earthquakes. In addition we have selected source zones to
describe the distribution of events with magnitudes between 4.5 and 6. Activity rates in each source zone have been determined from the historical/instrumental catalog.

We have developed a map of expected amplification due to site conditions (Figure 19), for 0.3 sec PSA. This map is based on the digital geologic map of surficial materials compiled by Carl Wentworth (1994). As a first step, we categorized the materials by their assumed shear-wave velocities: Bay Mud, class D ($V_s < 180$ m/sec); Quaternary alluvium, class C ($180-360$ m/sec); Quaternary and Tertiary sedimentary rock, class B ($360-750$ m/sec); Mesozoic and metasedimentary rock, including the Franciscan formation, class AB intermediate to classes A and B ($V_s$ about 750 m/sec); and igneous rock and schist, class A ($V_s > 750$ m/sec). Attenuation relations for each site class (A,B,C) were taken from the empirical results of Boore et al. (1993) derived from strong-motion data. For the Bay mud, the amplification factor was taken from the empirical relations between shear-wave velocity and amplification found by Borcherdt (1992). We expect that this map (Figure 19) will undergo significant changes after input from regional experts and further measurements of shear-wave velocity.

Figure 20 depicts our preliminary map of the ground motions with a 10% chance of exceedance in 50 years, for 0.3 sec PSA. The highest motions occur near the San Andreas and Hayward faults, in areas of Quaternary alluvium and Bay mud. Again the spatial distribution of ground motions is strongly affected by the location of alluvium and Bay mud. For downtown San Francisco the ground motions are about 130%g PSA (0.3 sec) for a class C site. Downtown Oakland has a probabilistic ground motion of about 170%g PSA (0.3 sec). Downtown San Jose has a probabilistic ground motion of about 150%g PSA (0.3 sec). In San Francisco, relatively large ground motions are found for the Marina district and the Embarcadero. Large probabilistic ground motions are found in the western edge of the Santa Clara Valley.

We can also express the probabilistic hazard in terms of the probability of exceeding 100%g PSA (0.3 sec) in 50 years. This is the nominal design value for buildings. Downtown San Francisco has about a 19% chance of exceeding 100%g PSA (0.3 sec) in 50 years. Downtown Oakland and San Jose have about a 29% chance of exceeding 100%g PSA (0.3 sec) in 50 years.

The probabilistic ground motions are higher where the ends of two faults are close or where two fault segments join. In the east Bay, these areas are at the southern end of the Hayward Fault where it is close to the Mission Fault in eastern Fremont and where the two segments of the Hayward fault join near San Leandro. If earthquakes are confined to each fault segment, then these locations will be more likely to experience strong shaking over a given period of time, compared to sites near the middle of the fault segments. Sites at the junction of faults or fault segments will experience shaking from both faults or segments, increasing the probabilistic hazard (see Discussion section).
parameter. Each discrete value of the parameter is given a weight, so that the weight of the final ground-motion value can be found for that combination of parameters. Average values and standard deviations can thus be calculated. In the Monte Carlo scheme, values of each parameter are specified by a distribution. For each run, values are taken for each parameter from this distribution. After many trials, the statistics (mean, median, standard deviation) of the probabilistic ground motions can be determined. Each of these approaches requires a subjective judgement of the range of parameters and their weights or distribution widths. In the past, these methods have been applied to ground motions at single sites. With the increasing speed of computers, maps of standard deviation of probabilistic ground motions will be made in addition to maps of mean probabilistic motions.

Conclusions

The development of new ground-motion hazard maps requires several tasks. The latest information on recurrence rates and attenuation must be incorporated into the national maps along with a consensus of experts in each region concerning source zonation. For regional maps, detailed site response information should be included, based on measurements of shear-wave velocity in surficial materials. When scenario earthquakes can be reasonably specified, deterministic maps of hazard should be made to compare to the probabilistic maps. In the future, results from 3-D simulations of basin response will also be incorporated into regional hazard maps.

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References

Figure 21 is a map of the deterministic ground motions calculated for the major faults in the region. For each site, we calculated the mean ground motions from characteristic earthquakes on each of the fault segments shown in Figure 18, using the empirical attenuation relations of Boore et al. (1993). We plot in Figure 21 the largest ground motions found for each site. We find mean ground motions between 150 and 250%g PSA (0.3 sec) near the San Andreas Fault for class C sites and mean ground motions of about 150%g PSA (0.3 sec) for class C sites in the east Bay along the Hayward Fault. As before, the pattern of ground motions is strongly influenced by site conditions.

The probabilistic ground motions (10% chance of exceedance in 50 years) are intermediate between the deterministic ground motions derived from the mean and those derived from the mean plus one standard deviation. Therefore, designing to these probabilistic ground motions should provide adequate protection for the occurrence of a large earthquake on either the San Andreas or Hayward faults.

Discussion: Probabilistic Versus Deterministic Methods and Uncertainty

One of the useful features of the probabilistic method is that it sums the hazard from all the source zones near the site of interest. The probabilistic method describes the probability of exceeding some ground motion from all possible source zones. Sites located near two active fault zones or two active areal source zones would have a higher probabilistic hazard than sites located at the same distance near only one of these source zones. Figure 22 shows a simplified example where two sites are situated near two active faults with similar rates of large earthquakes. Site A has a higher probabilistic ground motion (or hazard) than site B, since it will experience equal shaking from both faults over a period of time. Site B will not experience as strong shaking from the more distant fault, so it will have lower probabilistic hazard. Locations near two active sources (site A) simply have a higher likelihood of experiencing large ground motions over a fixed period of time than sites near only one active source (site B). This aspect of hazard is not conveyed in a deterministic hazard map, which simply gives ground motions expected for earthquakes on each fault. The deterministic approach would find that the two sites have equal deterministic ground motions. Given an earthquake on either fault, the two sites will experience equal ground motions. Clearly, for some applications, site A is more hazardous than site B. This is captured in a probabilistic analysis but not in a deterministic one.

Many parameters are required to produce probabilistic and deterministic hazard maps, and each parameter has a degree of uncertainty (see, e.g., McGuire, 1977). Two methods, the logic tree approach and the Monte Carlo method, have been used to characterize the uncertainty in probabilistic ground-motion estimates. In the logic tree approach, one calculates the results of using combinations of different values for each


California Division of Mines and Geology, 1967, "Geologic map of California, San Bernardino sheet."


Figure 1. Map of the 5 percent damped pseudo-acceleration spectral response, expressed in percent of the acceleration of gravity, with a 10 percent probability of exceedance in 50 years. The map values include estimates of variability in the attenuation of spectral acceleration and in fault rupture length.

(maps derived by Algermissen et al., 1991)
Figure 2. (a) Schematic map of seismic sources which contribute to hazard at site (circle), (b) log frequency of occurrence versus magnitude for each source zone, (c) attenuation relationships needed to calculate hazard at site.
Figure 3  Seismic source zones of the United States (exclusive of California) from Algermissen and others (1982) and historical earthquakes from Coffman and Von Hake (1973). Earthquake size is shown in terms of Modified Mercalli intensity (MMI). Significant earthquakes (MMI ≥ VII) are updated through 1990.
Annual Probability of Exceeding Ground Motion $U =$

\[
\text{annual rate of EQ's} \times \text{probability of exceeding } U \text{ if EQ occurs}\]

\[
\text{area of source zone}
\]

Sum over all magnitudes and locations within source zone

Figure 4: Calculating probabilistic hazard from single source zone.
Figure 5. Idealized distribution of ground motions at a site for an earthquake with a specified magnitude and distance. Variability is characterized by a log-normal distribution of ground motions about the mean value determined from averaging the logarithms of ground motion. Probability of exceeding ground motion $u_0$ is equal to the hachured area under the curve.
Downtown San Bernardino

Figure 6. Seismic hazard curve for a site in San Bernardino, California.
Figure 7. Uniform hazard spectra (10% probability of exceedance in 50 years) for selected cities
Figure 8. Comparison of normalized response spectra for eleven cities for 5 percent damping for an exposure time of 50 years with 10 percent probability of exceedance.

(from Leyendecker et al., 1994)
Figure 9. Comparison of complete response spectrum, approximate spectrum, and design spectrum for San Francisco.

(from Leyendecker and Algermissen, 1994; Leyendecker et al., 1994)
Figure 10. (left) Probabilistic map of Pacific Northwest from Algermissen et al. (1991), with one M8.25 earthquake on subduction zone every 500 years. (right) Probabilistic map with five M8.25 earthquakes on the subduction zone every 500 years (see Perkins and Hanson, 1993).
Spectral Acceleration, 0.3 Sec. period, 5% damping, 10 Percent Probability of Exceedance in 50 Years

Figure 11. Probabilistic hazard map for the central and eastern U.S. from Algernissen et al. (1991), values in percent g.
Figure 12  Probabilistic ground-motion hazard maps for the New Madrid region derived by (left) Toro et al (1992) and (right) Algermissen et al. (1990). Both maps show peak horizontal accelerations with a 90% probability of non-exceedance for 50 years. Values in map on left are in units of $g$, values in map on right are in percent $g$. Maps have different scales.
Figure 13. Source zones for California used in USGS hazard maps (derived from Thenhaus et al., 1980).
Spectral Acceleration, 0.3 Sec. Period, 5% Damping, 10 Percent Probability of Exceedance in 50 Years

Figure 14. Probabilistic hazard map for California and Nevada, from Algermissen et al. (1991). Values in percent g.
Figure 15. Map of San Bernardino Valley region showing three site classes used in hazard maps. Unshaded areas are recent alluvium, medium shaded areas are Pleistocene deposits, and dark shaded areas are bedrock. SAF denotes San Andreas Fault, SJF denotes San Jacinto Fault, and SB denotes downtown San Bernardino.
Figure 16. Preliminary probabilistic ground motion hazard map for San Bernardino Valley, for earthquakes on the San Andreas and San Jacinto Faults. Shading shows 0.3 sec pseudoacceleration response with 10% probability of exceedance for 50 years, in percent g. Lighter shades denote increased hazard. (from Frankel et al., 1994).

Figure 17. Preliminary deterministic (mean + 1 s.d.) ground motion hazard map for San Bernardino Valley, for earthquakes on the San Andreas and San Jacinto Faults. Shading shows 0.3 sec pseudoacceleration response for scenario earthquakes, in percent g. Lighter shades denote increased hazard. (from Frankel et al., 1994).
Figure 18  Map of the San Francisco Bay area showing segmentation models of faults used in producing the ground-motion estimates.
Figure 19. Preliminary map of site classes for the San Francisco Bay region.
Figure 20. Preliminary map of probabilistic ground motions for the San Francisco Bay region.
Figure 21. Preliminary map of deterministic ground motions for the San Francisco Bay region.
Figure 22. Map view of idealized example with two faults with equal recurrence times and magnitudes. Probabilistic hazard is higher for site A than site B; deterministic hazard is equal for both sites.
EARTHQUAKE GROUND MOTIONS IN THE NEAR SOURCE REGION

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Introduction

In this review, we discuss some of the many phenomena that can determine the nature of strong ground motion. Some of these phenomena have been inferred from the study of existing ground-motion records, and others are inferred from theoretical models of earthquake sources and wave propagation.

Perhaps the most common question posed in engineering seismology is; what is the size of ground motion at a given distance from an earthquake of a given magnitude? The answer to this question is surprisingly ambiguous. To illustrate, we show response spectra from horizontal ground motions recorded at distances near 50 km from shallow, crustal, strike-slip earthquakes of about magnitude 6.5 in Figure 1a (from Heaton et al., 1986). The distances actually vary from 36 km to 65 km and the magnitudes range from $M_w 6.1$ to $M_w 6.6$ and the spectra have been scaled to account for differences in these parameters. Even so, the largest motion is over 10 times larger than the smallest, and it is obvious that a wide variety of ground motions have occurred at a distance of about 50 km from magnitude 6.5 strike-slip earthquakes. The same degree of scatter can be seen in Figures 1b and 1c (from Heaton and Hartzell, 1989), where we show response spectra (unscaled) from shallow subduction earthquakes between magnitude 7.0 and 7.5 and observed at distances between 50 and 100 km, and between 101 and 150 km, respectively. This problem arises in virtually any data set that characterizes ground-motion amplitude as a function of distance and magnitude.

What phenomena are responsible for this large scatter, and what other parameters, besides distance and magnitude, can be used to predict near-source ground motions? The answer to this problem divides naturally into two classes of phenomena: those that are related to the rupture process (the source), and those that are related to the propagation of waves between the source and observer (path effect). Since the seismic velocity structure of the Earth is essentially constant in time, we expect that the effects of wave propagation can be included in any attempt to predict near-source ground motion. Predicting the rupture characteristics of future earthquakes may be impractical for the foreseeable future.

Effects due to wave propagation

The P- or S-waves that are radiated from a simple point dislocation in a homogeneous whole-space are described simply by a radiation pattern and an inverse-distance amplitude decay; the motion at the receiver is described by the time derivative of the dislocation history (ignoring, for the moment, near-field terms). For the sake of simplicity, ground-motion phenomena are sometimes interpreted assuming that our universe is indistinguishable from a homogeneous whole-space. In reality, seismic waves travel through a medium having a free
surface, systematic variations (usually increases) of velocity with depth, large-scale lateral variations (mountains and basins), small-scale lateral variations (scatterers), and dramatically different elastic properties at individual observation sites (soil conditions). Even for very simple sources, such as explosions, very complex wavetrains result from the propagation of waves through such complex media. When one is confronted with the variety of ground motions that often result from different stations recording the same earthquake, it is easy to despair at the seemingly uninterpretable variation in waveforms. However, careful studies of ground motions from well-recorded earthquakes and increasingly more realistic numerical models of wave propagation in complex structures are steadily improving our ability to understand and predict these complex waveforms. Studies of ground motions in densely instrumented regions have demonstrated that near-source waveforms, although complex, are quite coherent (Spudich and Cranswick, 1984; Hanks, 1975; Liu and Heaton, 1984).

In Figure 1, we showed the large variability of ground motions encountered at similar distances from similar-sized earthquakes. In Figure 2, we show response spectra from several different earthquakes, but recorded at the same sites (from Heaton and Hartzell, 1987). Clearly, the particular location of the recording site greatly influences the shape of the response spectra. What are some of the factors that control the path effect?

Near-site Soils

The 19 Sept. 1985 magnitude 8.1 subduction earthquake on the west coast of Michoacan, Mexico, has provided one of the most dramatic examples of the effects of localized site geology. Ground acceleration time histories for a variety of sites are shown in Figure 3 (from Singh et al., 1988). Ground motions in the vicinity of Mexico City are dramatically enhanced at periods of several seconds when compared with records from hard-rock sites closer to the earthquake. This phenomenon was previously noted by Zeevaert (1964), who interpreted it as the excitation of a fundamental shear modal vibration within the upper 50 meters of very low shear-velocity (about 60 meters/second) lacustrian deposits underlying parts of Mexico City. Tsai (1969) even generated synthetic ground motions that were similar to those that occurred in Sept. 1985.

Similar amplification has been proposed in other regions, such as the soft San Francisco Bay muds (Borcherdt, 1970). Differences in ground motions with depth in a 186 meter-deep hole in the San Francisco Bay muds were well modeled by assuming plane shear waves in a horizontally stratified medium (Joyner et al., 1976).

A recording of the ground motion in Niigata, Japan, from the 12 June 1964 magnitude 7.5 earthquake (Figure 4) provides another example of a dramatic site effect. High-frequency ground motions can only be seen in the first 6 seconds of the record after which there is an abrupt transition to much longer-period motion (from Ishihara, 1985). This site experienced extensive liquefaction during this event, and it is believed that the abrupt transition from short-period to long-period motion represents the onset of liquefaction. Liquefaction occurs during earthquakes when shaking causes materials to lose stiffness or strength, and it is usually associated with saturated, cohesionless soils. This is one of the most striking examples of
nonlinear, nonelastic site response. Ishihara et al. (1981) documented a less dramatic, but convincing, example of nonelastic site response in which they show an increase in pore fluid pressures (presumably caused by incremental volume changes in the soil) coincident with strong shaking on a hydraulic-fill island in Tokyo Bay during an earthquake on 25 Sept. 1980. However, the pore fluid did not reach values corresponding to complete liquefaction of the soil.

Basins

In a study of strong-motion data (principally from the 9 February 1971 San Fernando earthquake), Trifunac (1976) pointed out that larger ground velocities and displacements tend to occur more at sites characterized as "soft soils" than at sites on "hard rock." Curiously, this conclusion does not hold for the higher-frequency motions characterized by ground acceleration. This same tendency has also been observed in other strong-motion data sets (e.g. Joyner and Boore, 1982; Kawashima et al., 1984). Although this observation is often thought of as a local site effect, careful examination of the ground motions from the 9 February 1971 San Fernando earthquake leads to the conclusion that the excitation of surface waves (both Love and Rayleigh) within the San Fernando and Los Angeles basins is the origin of the large long-period ground motions observed at soil sites (Hanks, 1975; Liu and Heaton, 1984). In Figure 5 (from Liu and Heaton, 1984), we show ground-velocity records versus distance along an epicentral distance profile crossing two major basins, together with a profile of the subsurface geology. It is evident that long-period motions are predominantly surface waves that are excited within each basin, and that these surface waves do not propagate across the Santa Monica Mountains that separate these two basins. Vidale (1987) constructed finite-difference wave-propagation models to simulate the effects of this geologic structure, and he was able to reproduce many of the features seen in these records.

Basin structures may also serve to focus P and S body waves in much the same way that a lens focuses light (Rial 1984). Ihnen and Hadley (1986) have constructed three-dimensional body-wave ray-trace models of the Puget Sound region for the 29 April 1965 Seattle earthquake (magnitude 6.5). This earthquake occurred at a depth of 65 km beneath the Puget Sound and created a complex distribution of shaking intensities that can be explained, in part, by the focusing of body waves.

\( f_{\text{max}} \)

It has long been recognized that the average Fourier amplitude spectra of strong ground acceleration is approximately constant between frequencies of approximately 0.3 and 15Hz. It seems clear that the spectra fall off at low frequencies because of the finite duration of seismic sources. However, there have been several theories to explain the spectral fall off at high frequencies. Hanks (1982) has defined the term \( f_{\text{max}} \) to be the frequency at which an acceleration spectrum begins to fall off at high frequencies. We show an example of this spectral fall off in Figure 6 from Anderson and Hough (1984). They point out that the spectral decay at high frequencies seems to fit the functional form \( e^{-k f} \) better than it does the form \( f^{-\alpha} \). Their interpretation is that this functional form is easiest to explain by anelastic attenuation.
They further suggest that this occurs as the waves propagate through the uppermost several kilometers of the crust... This interpretation is reinforced by direct observations of body waves in deep boreholes (Malin and Walker, 1985; Hauksson et al., 1987). Although we presently favor the attenuation model for \( f_{\text{max}} \), there are other models in which \( f_{\text{max}} \) is explained as a source effect resulting from a characteristic distribution of fault slip (e.g. Aki, 1987).

The attenuation model for \( f_{\text{max}} \) may explain the surprising observation of very high-frequency, vertical-component P-waves that are sometimes recorded simultaneously with low-frequency, horizontal-component S-waves. Liu and Helmberger (1985) demonstrated that very strong shear-wave attenuation could be used to model high-frequency P-waves and low-frequency S-waves that were observed in the strong motions of the 15 Oct. 1979 Imperial Valley earthquake and its aftershocks (Brady et al., 1982; Anderson and Heaton, 1982).

**Eastern US vs. Southwestern US vs. Subduction Zone**

The propagation of seismic waves through the crust is an important factor that influences the nature of ground motions at any site. Although the detailed description of the physics necessary to fully solve this problem may be very complex, simple distance attenuation laws have been developed to approximate the effect of this complex wave propagation. We compare some representative attenuation laws for response spectral velocity at 1 second and for different regions in Figure 7. Because of the relatively large data set for earthquakes up to magnitude \( M_\text{L} = 7 \) from the southwestern United States (Joyner and Boore, 1982), an average attenuation law is fairly well determined for this region and values for both "rock" and "soil" sites are given (the scatter of the data about that law is very large). In contrast, there are very few strong-motion data available from the eastern US and consequently there is considerable uncertainty regarding eastern US ground motions. However, it seems clear that earthquakes in the eastern US have been accompanied by relatively large areas of moderate shaking intensity (Nuttli, 1973). It has been proposed that regional differences in the characteristics of the phase \( L_g \) is the primary reason that eastern US ground motions may decay with distance less rapidly than in the southwestern US (Shin and Hermann, 1987). The eastern US attenuation curve in Figure 7 is from Boore and Atkinson (1987) who assumed that eastern US earthquakes are similar to those in the southwestern US and hence that the ground motions are very similar at close distances.

The distance attenuation law derived using ground motions from large Japanese subduction earthquakes (Kawashima et al., 1984) is surprisingly different from the attenuation law for the southwestern US. The relatively gentle distance attenuation law is a reflection of the fact that large ground motions have been observed at surprisingly large distances from large subduction earthquakes (Heaton and Hartzell, 1987). Curiously though, ground motions for Japanese earthquakes of magnitude less than 7.0 seem roughly comparable to those that have been observed in the southwestern US (Heaton et al., 1986). At this point, we do not know whether these observations indicate that the distance attenuation law is fundamentally different for subduction zones, or whether the distance attenuation law changes for very large subduction earthquakes as the source dimensions become large.
Fault Zones as Waveguides

There is evidence that major fault zones, such as the San Andreas, are associated with a zone of relatively low seismic velocities that is as much as several kilometers wide (Feng and McEvilly, 1983). Cormier and Spudich (1984) have modeled the propagation of high-frequency body waves from earthquakes occurring in low-velocity fault zones. They find that the low-velocity region serves as a waveguide, and amplification of as much as a factor of 10 can occur for body waves observed at sites within the fault zone. Cormier and Beroza (1987) suggest that the high accelerations observed on the Calaveras fault during the 24 April 1984 Morgan Hill, California, earthquake may have been caused by focusing of energy along the fault zone.

Effects due to the Seismic Source

We use the term source effects to signify those phenomena that result from the distribution of slip (in time and space) that occurs during earthquakes. Although this distribution may be quite complex, it is often convenient to assign a single number to an earthquake to indicate the earthquake’s size. This number is usually a magnitude, a moment, or sometimes an energy. It is beyond the scope of this paper to discuss the various methods of earthquake quantification, but the subject is discussed in a general way by Kanamori (1978), Báth (1981), and Heaton et al. (1986). Although the quantification of an earthquake source with a single number may be convenient, it cannot hope to provide sufficient information to allow a specific prediction of the seismic waves that will be radiated from the source region. As an example of the importance of the seismic source on strong ground motions, we show ground motions in Figure 8 for two different earthquakes that were recorded at the same site in El Centro, California. These earthquakes have both been assigned surface-wave magnitudes ($M_s$) of 6.5, and they occurred very close to each other, perhaps on different segments of the same fault system. A close inspection of the records reveals striking similarities and differences; the long-period surface-wave parts of the record are very similar, and yet the higher frequency velocity and acceleration records are very different. Although these records are not yet fully understood, it seems likely that the large pulse seen at the beginning of the record from the 1968 Borrego Mtn. earthquake is a direct effect of the rupture process.

We now discuss some of the ways that the seismic source influences ground motions.

Directivity

If an earthquake rupture propagates along a fault at some velocity, then observers that lie in the direction of propagation will record shorter duration (and hence higher amplitude) ground motions than observers located opposite the direction of rupture propagation. This phenomenon is called directivity, and it is similar to the Doppler effect. One of the most convincing examples of this phenomenon is from the 15 October 1979 Imperial Valley, California, earthquake. Low-pass-filtered ground velocities are displayed on a map showing the location of the recording stations and the causative fault in Figure 9 (from Archuleta, 1984). Rupture propagated to the northwest from the epicenter, located just south of the Mexico - United States border. Although the stations BCR at the southeastern end of the
rupture and E06 at the northwestern end of the rupture are located approximately symmetrical with respect to the fault plane, E06 (and adjacent sites) recorded a large pulse of ground motion that does not appear at BCR. Hartzell and Helmberger (1982), Olsen and Absel (1982), Hartzell and Heaton (1983), and Archuleta (1984) have all successfully modeled these records using directivity. Strong motion data from the 24 January 1980 Livermore Valley earthquake provides another dramatic example of directivity (Boatwright and Boore, 1982). Heaton and Helmberger (1979) have also concluded that the large velocity pulse observed on the famous Pacoima Dam recording of the 9 February 1971 San Fernando earthquake is the result of directivity. Although these are some of the most dramatic examples of directivity, this phenomenon is certainly fairly common, since rupture propagation along finite faults is endemic to earthquake sources.

Asperities and Barriers

The 15 October 1979 Imperial Valley earthquake occurred in a relatively simple geologic structure (the Salton trough), and it was well recorded in the near-source region (Figure 9). Because of these factors, there has been considerable success in the modeling of the rupture process of this earthquake (Hartzell and Heaton, 1983; Archuleta, 1984). In Figure 10a, we show the final slip distribution for the preferred rupture model of Hartzell and Heaton (1983). Although the total rupture length is nearly 40 km, a large part of the slip is localized to a patch approximately 6 km long. Such patches of higher slip are referred to as asperities and the asperity in the models of the Imperial Valley earthquake plays a central role in the simulation of the observed ground motions. Furthermore, Spudich and Cranswick (1984) conclude that this asperity is the probable source of much of the very high-frequency P-wave motions observed for this earthquake.

Although the geometry of the fault, geologic structure, and recording sites was not quite as advantageous for study of the source of the 24 April 1984 Morgan Hill earthquake, there are clear indications that there were at least two asperities that strongly affect the nature of the ground motions for this earthquake. We show the preferred slip-distribution model of Hartzell and Heaton (1986) in Figure 10b. They report that many features of the observed ground motion are explained by the failure of a small asperity near the hypocenter followed by a much larger asperity failure about 12 km to the south.

Asperities have also been introduced into models for large subduction earthquakes. Although the asperities are on a much larger scale (tens of kilometers), there is evidence that the magnitude 8.1 Michoacan, Mexico, earthquake of 19 September 1985 may be thought of as the rupture of two separate large asperities. These two events can be seen in both the strong ground motions (Anderson et al., 1987) and in the teleseismic P-waves (Houston and Kanamori, 1986). The correspondence between asperities inferred from teleseismic P-waveforms and ground motions recorded in the near-source region is discussed for a number of large subduction earthquakes by Heaton and Hartzell (1989). Kanamori (1986) and Lay et al. (1982) review evidence for asperity distributions in large subduction earthquakes.
Aki et al. (1977) and Das and Aki (1977) propose that the rupture along faults may be interrupted by patches that do not rupture in any given earthquake. The patches are called barriers, and if the rupture terminates abruptly, then high-frequency "stopping phases" will be generated. Papageorgiou and Aki (1983a,b) present barrier models for a number of historic earthquakes. One feature of these models is that they constitute an alternative explanation for the $f_{max}$ phenomena discussed previously in this paper.

Roughness

Ground acceleration time histories in the near-source region have many similarities to random time series (Jennings et al., 1968; Boore, 1983). However, models that specify uniform rupture properties along a fault tend to produce motions that have high-frequency arrivals that result only from the initiation and termination of rupture. Haskell (1966) and Aki (1967) introduced source models in which the rupture process is considered to include statistical irregularities with prescribed correlation distances. Andrews (1981) demonstrated a rupture model in which the stress drop on a fault plane is assumed to be both random (not necessarily uniformly random) and self-affine in space (changes in length scale do not change the statistical nature of the distribution). The spatial distribution of fault slip for a model of this type is shown in Figure 11 (from Andrews, 1981). Representative records of ground velocity and displacement that result when such a rupture model is embedded in a homogeneous whole-space are also shown. Boatwright and Quin (1986) discuss the behavior of similar models, but which have the added feature of spontaneous rupture initiation for any point on the fault. Both Andrews (1981) and Boatwright and Quin (1986) demonstrate stress-drop distributions that produce radiated ground motions having constant Fourier amplitude spectra of ground accelerations at frequencies short when compared with the overall rupture duration. We presently do not know how the statistical properties of faults change regionally, or how such variations may affect near-source ground motions.

Stress Drop

The term stress drop is both very common in seismology and often quite confusing. Intuitively, it is easy to imagine that the level of stress driving a dislocation should have an important effect on the strength of the radiated seismic waves. However, direct measurements of changes in stress during an earthquake are virtually impossible, and hence this parameter must be deduced through the use of models and indirect measurements. Consequently, there are many different definitions for "stress drop," which would all converge to the same physical parameter if all of the models were appropriate to simulate the Earth. Although it is beyond the scope of this review to comment on individual ways that "stress drop" is measured, we briefly comment on the most common application of this term in strong-motion seismology.

A widely used model for the general quantification of earthquake sources is the $\omega$-squared model of Aki (1967) and Brune (1970, 1971). In this model, ground motions from earthquakes have a Fourier amplitude spectrum given by $S(\omega) = A_0 \omega_c^2 / (\omega_c^2 + \omega^2)$, where $\omega_c$ is defined as the spectral corner frequency, and the low-frequency ($\omega < \omega_c$) level $A_0$ is
proportional to the seismic moment $M_0$. The corner frequency for S-waves is given by
\[ \omega_c = 0.49\beta^2 \pi (\Delta \sigma / M_0)^{1/3} \]
where $\Delta \sigma$ is the stress drop and $\beta$ is the S-wave velocity at the source. In this model, the corner frequency is assumed to be inversely proportional to the duration of the S-wave, which is, in turn, assumed to be proportional to the dimension of the rupture. The stress drop is then assumed to be proportional to the moment divided by the cube of the source dimension. In practice, this stress drop is measured by fitting asymptotes to log-log spectral amplitude plots, and thus this definition of stress drop gives a measure of the relative amplitude of long- and short-period seismic radiation. The $\omega$-squared spectral model, together with the assumption of a stress drop that is independent of seismic moment, has been remarkably successful in fitting observed earthquake ground motions over a wide range of earthquake sizes (Hanks, 1977).

If the $\omega$-squared model is assumed, and if high-frequency motions are assumed to have the same statistical characteristics as random vibrations, then the time-domain amplitude of high-frequency ground motions can be shown to be proportional to $M_0 \omega_c^{5/2}$ (Hanks, 1979), which in turn is proportional to $M_0^{1/6} \Delta \sigma^{5/6}$. Hanks and McGuire (1981) studied acceleration data from 16 California earthquakes and concluded that these earthquakes had stress drops (sometimes referred to as rms stress drop) of about 100 bars with a standard deviation of a factor of two. In other words, the long-period motions averaged over many records could be used to predict (within a factor of two and a confidence of 85%) the average over many records of the high-frequency motions. As it is used in this sense, stress drop (rms) is really a spectral scaling parameter that gives a measure of the ratio of the high- and low-frequency spectral levels.

As we previously mentioned, stress drop is measured in many ways, none of which are direct measurements of the actual stress drop on the rupture surface. However, the inference that large ground motions often result from localized regions of high slip, and hence high stress drop (asperities), strongly implies that the actual stress drop during a rupture influences the strong ground motions. One question of considerable concern is whether plate-boundary earthquakes (interplate, e.g. those in California) have systematically lower stress drops than earthquakes away from plate boundaries (intraplate, e.g. those in the eastern United States). Kanamori and Anderson (1975) studied seismic moment as a function of fault dimension (defined by aftershocks), and they concluded that intraplate earthquakes have average stress drops (sometimes referred to as static stress drop) that are about twice as large as interplate earthquakes. Kanamori and Allen (1986) show that faults with long recurrence intervals are likely to have shorter rupture lengths for similar-sized ($M_s$) earthquakes than faults with short recurrence intervals (Figure 12). Both of these observations would imply stronger shaking at the source for eastern US earthquakes. However, Somerville et al. (1987) compared teleseismic P-waveforms from moderate-sized earthquakes from the eastern and western US and concluded that systematic differences do not exist. The eastern US ground-motion estimates of Boore and Atkinson (1987) shown in Figure 7 were calculated assuming an $\omega$-squared model with stress drops of 100 and 50 bars in the eastern and southwestern US,
respectively. However, they also assume that site amplification at rock sites in the eastern US and southwestern US are 1.0 and 2.0, respectively. Thus, epicentral ground motions in the eastern and southwestern US nearly coincide in Boore and Atkinson's (1987) model. At present, there still seems to be some uncertainty about the relationship between static stress drop (a function of seismic moment and overall rupture length) and rms stress drop (a function of the shape of the Fourier amplitude spectrum of body waves).

Radiation Pattern

Haskell (1964) showed that a point shear dislocation in an elastic medium is indistinguishable from a point double-couple force system. Furthermore, the amplitude and polarity of waves radiated from a point double couple varies with the relative orientation of the source and the receiver, and the relationships that describe this variation are referred to as the radiation pattern. Liu and Helmberger (1985) discuss the strong ground motions produced by a relatively simple aftershock of the 15 October 1979 Imperial Valley earthquake, and they show convincing evidence of the effect of radiation pattern on the amplitude of motions at periods greater than 1 sec. Although there is a clear theoretical basis for the importance of radiation pattern, there are also several examples of instances where relatively large ground motions are observed at locations where a simple application of seismic radiation pattern would predict very small amplitudes. In particular, large high-frequency P-waves have been observed at stations located directly along the fault strike for vertical strike-slip earthquakes, an orientation for which the radiation pattern would predict zero amplitude. Cormier and Spudich (1984) suggest that these observations may be due to the lateral refraction of waves within the low velocities that exist along fault zones (see the earlier section on fault zones as waveguides).

Aspect Ratio

The aspect ratio of a rupture is the ratio of its length to its width. Although the rupture lengths and widths are probably roughly comparable for small- and moderate-sized earthquakes, there may be a very large variation in the rupture aspect ratio for very large earthquakes. For instance, the great shallow strike-slip California earthquakes of 1906 and 1857 (about M 8) had rupture lengths of hundreds of kilometers, whereas their rupture widths were probably less than twenty kilometers. However, M 8 earthquakes from subduction zones may have roughly comparable rupture lengths and widths of about 100 km. It seems likely that strong ground motions resulting from a long narrow rupture are different from those that result from a very wide rupture. Perhaps the aspect ratio of a rupture is one of the reasons that relatively large ground motions are observed at large distances from large subduction earthquakes.

Near-Field

"Near-field" has two separate and very different meanings in seismology. Although it is often used to describe the geographic region within tens or even hundreds of kilometers of an earthquake, it also is used to signify a particular type of ground motion that is best observed very close to the source. We prefer to use the term near-field only in the context of the second
meaning and to refer to the geographic region as near-source. In the simple problem of a point dislocation embedded in a homogeneous medium, the complete solution can be expressed in a simple way as the sum of several terms that decay with distance with different integer powers of distance (Harkrider, 1976; Heaton, 1978). Terms that decay inversely with distance produce motions that are the time derivative of the dislocation and these parts of the solution are called the far-field waves. Terms that decay inversely as distance squared (or higher powers) produce motions that are proportional to (or time integrals of) the dislocation, and these parts of the solution are called the near-field waves. Many of the generalizations that are made for far-field waves do not apply to near-field waves, and special care must be taken in their interpretation. Heaton (1978) and Heaton and Helmberger (1979) discuss near-field waves in some detail.

Strong ground motions recorded above the rupture surface of the 19 September 1985 Michoacan, Mexico, earthquake provide one of the most dramatic examples of near-field waves yet observed. Vertical ground-displacement records (obtained from doubly integrated acceleration records) are shown in Figure 13 (from Anderson et al., 1987). The large ramp-like displacement that results in a net vertical displacement of the ground is almost certainly the result of near-field waves, and the total offset observed in these records is compatible with changes in sea level observed near these stations. Even larger near-field waves can be inferred for other great earthquakes. For example, dislocations of 7 meters have been inferred along some sections of the San Andreas fault during the 1906, California, earthquake (Thatcher, 1975), and we can thus conclude that near-field waves adjacent to the fault were at least 3.5 meters. Even larger horizontal surface displacements of at least 20 meters were observed for some regions during the 1964 Alaskan earthquake (Plafker, 1965) (Mw 9.2). Estimating the time scale over which such large static displacements occur is still somewhat problematic, but a simple model of spontaneous slip resulting from a stress of 100 bars implies dislocation velocities of about 1 meter/second (Brune, 1970).

Discussion

In this review we have shown examples of many different phenomena that affect the nature of ground motions in the near-source region. It is not surprising that such a diversity of ground motions have been observed at comparable distances from earthquakes of comparable size. Although the problem of predicting ground motion is very complex, many of these phenomena can be anticipated - particularly path effects. Great progress has been made toward a better understanding of the physics of these phenomena, and we anticipate that future research will allow us to produce more accurate estimates of the nature of the ground motions that can be expected at particular sites. We are particularly optimistic that the effects of wave propagation can be anticipated through the study of ground motions from relatively numerous small earthquakes and through the modeling of waveforms within complex geologic structures.
References


Figure 1. (a) Response spectra (3% damped) for horizontal components of 15 records from strike-slip earthquakes that are scaled to a distance of 50 km and a magnitude of 6.5 (from Heaton et al., 1986). (b) Response spectra (5% damped) of horizontal components (not scaled for distance or magnitude) of 7 records from sites ranging between 50 and 100 km from shallow subduction earthquakes having magnitudes between 7.0 and 7.5 (from Heaton and Hartzell, 1987). (c) Same as (b), except that sites range between 101 and 150 km. These composite plots show the type of scatter that is typical of ground motions recorded at a given distance and magnitude.
Figure 2. Response spectra (5% damped) of horizontal components of ground motion from several different earthquakes recorded at the same site. Notice that features in the response spectra are similar from one earthquake to another, thus indicating the importance of path effects (from Heaton and Hartzell, 1987).
Figure 3. EW component of acceleration from the 19 Sept. 1985 Michoacan, Mexico earthquake from selected sites. Records are shown in order of increasing distance from the rupture area. Sites CU1P, CUMV, SXVI, TACY, SCT1, CDAF, CDAO, TLHD, and TLHB are all in Mexico City. Note the variation in ground-motion characteristics for this earthquake (from Singh et al., 1987; copyright Seismological Society of America).
Figure 4. Ground acceleration from a site in Niigata, Japan, that experienced liquefaction. The change in frequency content of the record at about 8 seconds may be due to a loss of stiffness (liquefaction) in the materials beneath the site (Ishihara, 1985; figure is published by the National Academy of Sciences, 1985).
Figure 5. Transverse component of ground velocities measured during the 9 Feb. 1971 San Fernando earthquake. Records are plotted as a function of epicentral distance along a profile running south across the San Fernando Valley and the Los Angeles basin. The corresponding free-surface and basement-surface profiles are shown to the left. The dashed line indicates the possible phase arrival of surface waves. Notice that apparent surface waves seen within the basins do not appear to propagate across the Santa Monica Mountains (from Liu and Heaton, 1984).
Figure 6. Fourier amplitude spectrum of the N85°E component of ground acceleration recorded at Cucapah from the Mexicali Valley, Mexico, earthquake of 9 June 1980 (Mw 6.2). (A) Log-log axes. (B) Linear-log axes. Plot shows that spectral amplitudes decay at very high frequencies in an exponential manner typical of anelastic attenuation (from Anderson and Hough, 1984; copyright Seismological Society of America).
Figure 7. Comparison of distance attenuation laws derived from different regional data sets. The southwestern US curves are derived chiefly from recordings of earthquakes smaller than M 7 (Joyner and Boore, 1982); the eastern US curve is derived assuming the southwestern US ground motions in the epicentral region together with a distance attenuation law derived from small-earthquake data (Boore and Atkinson (1987); and the subduction-zone curves are derived chiefly from large (magnitude greater than 7) Japanese earthquakes (Kawashima et al., 1984).
Figure 8. North component of ground motion for two M 6.5 Borrego, California, earthquakes as recorded at El Centro. Note that the 1942 records are plotted on an amplitude scale half as large as that used to plot the 1968 records. Although these motions were recorded at the same station and were from comparable earthquakes, the 1968 velocity records are very different from those in 1942. However, similar long-period surface waves were recorded for both events (from Heaton et al., 1986).
Figure 9. Low-pass-filtered ground velocities (N37°W) recorded during the 15 October 1979 Imperial Valley, California, earthquake. The rupture propagated from its southern epicenter (asterisk) northwestward along the Imperial fault. Stations located along the fault toward the northern end of the rupture recorded much larger ground velocities than did more southern stations (from Archuleta, 1984; published by the American Geophysical Union).
Figure 10. Map showing slip distributions (in cm.) inferred for (a) the 15 October 1979 (M6.5) Imperial Valley earthquake (Hartzell and Heaton, 1983) and (b) the 24 April 1984 (M 6.2) Morgan Hill, California, earthquake (Hartzell and Heaton, 1986). Asperities (localized regions of large slip) are thought to play an important role in the radiation of seismic waves from these events. The asterisk in (b) shows the hypocenter.
Figure 11. Theoretical model of fault slip assuming that stress drop varies randomly on the fault plane (defined by the X and Y coordinates). Representative body-wave records (both displacement and velocity) are shown for such a model (from Andrews, 1981; published by the American Geophysical Union).
Figure 12. Relation between surface-wave magnitude, $M_S$, fault length, and earthquake recurrence time. The solid lines indicate the trends for constant stress drop. This is evidence that faults with long recurrence intervals may have earthquakes with higher stress drops and perhaps larger high-frequency motions (from Kanamori and Allen, 1986; copyright by the American Geophysical Union).
Figure 13. Ground displacements (from doubly integrated accelerometer records) observed at three sites directly above the 19 Sept. 1985 Michoacan (MW 8.1) earthquake. The large static offset is real and was also observed in changes of sea level adjacent to the recording sites. This is one of the most dramatic examples of near-field waves recorded to date (from Anderson et al., 1986; copyright by the American Association for the Advancement of Science).
MODELLING GROUND MOTIONS IN THE NEAR-FIELD OF RUPTURING FAULTS

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Introduction

Strong ground motions within one or two kilometers of a rupturing fault pose an important problem for seismic engineering. Figure 1 shows a set of intensity estimates compiled by Borcherdt et al. (1975) for more than 900 sites on Franciscan formation in San Francisco and the Bay Area during the 1906 earthquake. While the observations of the highest intensities (Very Violent) along the fault trace might be disregarded as many of these intensities were associated with the observed surface faulting rather than derived from any measure of ground shaking, the 236 observations of the next highest intensities (Violent) at distances from 0.3 to 1.6 km from the fault explicitly represent damage and intensities associated with ground shaking.

Furthermore, these strong ground shaking effects persist along the surface traces of faults in the absence of surface faulting. Landslides, thrown rocks, and topped trees occurred throughout the epicentral area of the 1989 Loma Prieta earthquake (Kiefer, 1993). Estimates of intensity for the Loma Prieta earthquake, determined by evaluating the damage to building stocks in 30 different structural categories and corrected for the amplification effects of the surficial geology, describe narrow zones above the buried rupture where the equivalent 1906 intensities were Very Violent and Violent (Perkins, oral communication, 1993). Because no recent large Californian earthquakes have occurred on faults directly underlying metropolitan areas, however, the results of such intensity surveys are still regarded as speculative.

While these damage and intensity studies indicate that ground motions are “violent” near the fault trace, the set of accelerograms obtained near rupturing faults is sparse and generally problematic. The Pacoima Dam recording of the 1971 San Fernando earthquake (PGA = 1.25 g), the Coyote Lake Dam recording of the 1984 Morgan Hill earthquake (PGA = 1.30 g), and the Lucerne Valley recording of the recent 1992 Landers earthquake (PGA = 0.88 g) represent some of the largest horizontal accelerations ever recorded. In general, the sampling of acceleration as a function of distance from the fault is relatively complete for only one earthquake, the 1979 Imperial Valley earthquake. The El Centro accelerograph array extended some twenty kilometers on either side of the Imperial fault. Near the fault, El Centro 6 recorded a vertical PGA of 1.75 g, while the horizontal PGA was 0.46 g, the same as the horizontal PGA at El
Centro 7. A horizontal PGA of 0.78 g was recorded at Bond's Corner, one kilometer from the Imperial fault.

In the face of this weak sampling of both intensity and ground motion near rupturing faults, it is critical to consider appropriate methods for modelling these near-fault ground motions. In general, it is necessary to determine which physical models and approximations are adequate. In this draft of this paper, we will discuss the first of two antithetic "source" models for the near-fault ground motions. This model was analyzed in detail by Luco and Anderson (1983) and Anderson and Luco (1983): it treats the faulting process as a uniform dislocation that propagates along an infinite fault at a constant velocity. The second model is an extension to the near-field of Boatwright's (1982) model for far-field acceleration. These two models contain complementary parts of the seismic radiation from a rupturing fault. Comparing the fields radiated by these two antithetic models yields useful insights into the character of the near-fault motions.

In addition to these two "source" models, we will also discuss, in a later draft, a recent "propagation" model for vertical strike-slip faults derived theoretically by Li et al. (1990) and tested by analyzing near-fault recordings of aftershocks of the 1992 Landers earthquake by Li et al. (1994). In this model, the ground motions near the fault are significantly amplified (or focussed) by the low velocity structure associated with the 1-2 km wide fault zone itself. These low velocity materials are also strongly attenuating, however. By combining these source and propagation characteristics appropriately, we can determine a range of strong ground motion sufficient for modelling the Violent and Very Violent intensities observed near rupturing faults.

A Coherent Model for Fault "Fling"

Luco and Anderson (1983) analyze the ground motions radiated by a very coherent fault model: in their model, the rupture process is an instanteous slip dislocation that uniformly propagates down an infinitely long fault. This approximation ensures that the motions they synthesize are exactly the response of the elastic medium to the rupture passage. Recent seismological and engineering discussions of the rupture process have incorporated the term "fling" to describe this phenomenon (Bolt, oral communication, 1983). In the somewhat arcane terminology of the Green's functions used to model motions resulting from slip dislocations, the Luco and Anderson (1983) model contains the "near-field" and "intermediate-field" terms of the Green's function (Aki and Richards, 1980).

We will restrict our discussion of their results to the case of vertical strike-slip faulting considered in Anderson and Luco (1983). Figure 2 shows the geometrical description of the model and the relevant parameters. The fault is embedded in a half-space, extending from $x = -\infty$ to $x = \infty$ along strike and from $z = z_d$ to $z = z_u$. 
in depth. The rupture velocity along strike is \( c_1 \). The "inclination" of the rupture front is controlled by the velocity \( c_2 \): if \( c_2 \) is positive, the shallow edge of the rupture leads the deeper edge, while if \( c_2 \) is negative, the shallow edge of the rupture lags the deeper edge. In the examples from Anderson and Luco (1983) that we will present and discuss, the slip dislocation is assumed to occur instantaneously.

Examples of the seismic radiation from this uniformly propagating rupture, calculated by Anderson and Luco (1983), are plotted in Figures 3 and 4 as a function of the normal distance from the fault trace \( y \). The top of the buried fault is at \( z_u = 2 \) km while the bottom is at \( z_d = 10 \) km. The slip dislocation is \( \Delta_o = 100 \) cm. These parameters are chosen so that the resulting model resembles the 1979 Imperial Valley earthquake. For the synthetics plotted in Figures 3 and 4, however, \( c_1 = 0.919\beta \) has been set nearly to the Rayleigh wave velocity, or \( c_r \approx 0.919\beta \) for a Poisson solid, where \( \beta \) is the shear wave velocity. The Rayleigh wave velocity is the "terminal" velocity for their model: this choice maximizes the accelerations radiated by their model. The vertical rupture velocity \( c_2 \) has been set to \( \infty \) so that the rupture front is vertical.

The synthetic displacements plotted in Figure 3 eloquently realize the concept of fault "fling." The largest motions occur on the transverse component \( u_y \). The passage of the rupture front (at \( t = 0 \) s) generates a one-sided transverse displacement pulse of 2-3 s duration, whose amplitude and shape is relatively constant near the fault, that is, for \( y < 5 \) km. The peak displacement (\( \Delta_o/3 \approx 30 \) cm) scales linearly with the fault slip. The peak displacement of the transverse pulse also depends on the rupture velocity: if rupture velocities in the range \( 0.8\beta < c_1 < 0.85\beta \) are used instead of 0.9184\( \beta \), the peak displacement is reduced by 30%.

The synthetic ground velocities are plotted in Figure 4. The symmetric displacement pulses of Figure 3 are differentiated to antisymmetric velocity pulses: the apparent pulse width is still about 3 s. The peak velocities of \( \approx 25 \) cm/s are relatively constant near the fault, but the zone is somewhat more narrow, \( y < 2 \) km. The peak velocity of the transverse pulse depends strongly on the rupture velocity: if rupture velocities in the range \( 0.8\beta < c_1 < 0.85\beta \) are used instead of 0.9184\( \beta \), the peak velocity would be reduced by a factor of 2-3.

While the general calculation of these near-field motions requires a numerical integration, Luco and Anderson (1983) obtain a closed form expression for the SH component of the transverse ground velocity exactly above the buried fault (at \( y = 0 \))

\[
\dot{u}_y^{SH}(0,t) = \frac{\Delta_o}{\pi t} \left[ \Phi(z_d, c_1, t) - \Phi(z_u, c_1, t) \right]
\]

where

\[
\Phi(c_1, z, t) = \frac{1}{b} \left( 1 - \frac{1 - b/2}{\sqrt{1 - b}} \right) \quad \text{and} \quad b = (c_1/\beta)^2 \left[ 1 - (\beta t/z)^2 \right].
\]
As Figures 3 and 4 demonstrate, the transverse velocity and displacement are approximately constant for \( y < z_u \), so that Luco and Anderson's (1983) expression provides a useful approximation for the transverse velocity near the fault trace.

If we integrate the transverse velocity to displacement and then integrate the resulting transverse displacement pulse, we obtain a simple relation for the area of the near-field pulse

\[
\int u^H_y(0, t) \, dt = \Delta v \Xi(z_d, z_u, c_1)
\]

that requires the double integral

\[
\Xi(c_1, z_d, z_u) = \int_{-\infty}^{\infty} \int_{-\infty}^{t} \left[ \Phi(z_d, c_1, \tau) - \Phi(z_u, c_1, \tau) \right] \frac{dr}{\pi \tau} \, dt
\]

to be evaluated numerically. This relation bears some analogy to the relation between the far-field displacement pulse and the seismic moment. The pulse area is independent of the form for the near-surface slip function and the inclination of the rupture front. Generally, \( \Xi(c_1, z_d, z_u) \approx 0.3 - 0.4 \text{ s} \). The pulse area depends strongly, however, on the rupture velocity, \( c_1 \), as shown in the contour plot in Figure 5, but not on the depth that the fault is buried, for \( z_u < 2 \text{ km} \).

If the pulse area can be adequately resolved from near-field data, estimates of \( \Delta v \) and \( z_d \) might be used to determine \( c_1 \). It is useful to note, following Anderson and Luco (1983), that the rupture velocity appropriate for this modelling is the average rupture velocity as the rupture passes the station. If the rupture front is transonic (\( \beta < c_1 < \alpha \)) or if the rupture front changes velocity rapidly as it ruptures past the station, the model approximations (constant slip and constant rupture velocity) break down.

Anderson and Luco (1983) complied peak acceleration, velocity, and displacement as a function of distance from the fault for three different fault models, reprinted as Figure 6. All three models have slightly slower rupture velocities (\( c = 0.86 \beta \)) than the model used to generate the synthetics plotted in Figures 3 and 4. The fault “fling” attenuates rapidly as the observer moves away from the fault. At distances greater than \( z_u \), the transverse peak acceleration and peak velocity attenuate as \( y^{-2} \), while the peak displacement attenuates as \( y^{-1} \). Moreover, the “width” over which the peak motions are constant increases approximately from \( z_u \) to \( 2z_u \) to \( 4z_u \) for the acceleration, velocity, and displacement, respectively.

The last parameter of concern is the “inclination” of the rupture front. Anderson and Luco (1983) analyzed the effect of this inclination as shown in Figure 7. Varying the inclination of the rupture front changes the phase of the pulse: if the upper edge of the rupture lags behind the lower edge, as might be reasonably expected for a horizontal rupture in a crust where the wave velocities increase with depth, the phase lag of the
pulse increases so that the peak velocity occurs on the return. In contrast, if the upper edge of the rupture leads the lower edge, the phase of the pulse decreases and the peak velocity occurs during the initial motion, or fling. We note that the closed form for the transverse velocity given in equation (1) cannot be readily generalized to accommodate an inclined rupture front, and the original numerical integration of Luco and Anderson (1983) should be used.

Anderson and Luco (1983) also show that the peak velocity decreases by about a factor of two if the rupture fronts are steeply inclined (that is, \( |c_2| < |c_1|/2 \)). The effect on the peak displacement is less pronounced, although clearly, the more inclined the rupture front, the broader the displacement pulse. Both the inclination of the rupture front and the shape of the slip function strongly affect the shape of the transverse velocity and displacement pulses, and must be simultaneously constrained to fit near-fault observations.

Fitting Near-Field Displacement and Velocity

Anderson and Luco (1983) conclude their “Parameteric Study of Near-Field Ground Motion for a Strike Slip Dislocation Model” by attempting to fit the data obtained from the El Centro array during the 1979 Imperial Valley earthquake. Figure 8 shows their comparison of the transverse velocity pulses recorded at stations 6 and 7 of the array with a synthetic pulse determined from a model specified using the parameters listed in the Figure caption. The depth of the upper edge of the fault \( (z_u = 1.0 \text{ km}) \) is set equal to the normal distance from the fault, \( y \). The rupture velocity is relatively slow, \( c_1 = 2.0 \text{ km/s} \). The synthetics, while matching the shape of the ground velocities, markedly underestimate the amplitude. Further comparisons of data to their model synthetics show that this underestimate is characteristic for this earthquake.

There are two elements which contribute to this misfit. The first is that the rupture process for this earthquake is significantly non-uniform: as Olson and Apsel (1982) and Archuleta (1984) have shown, the rupture velocity in this earthquake accelerated to transonic velocities as it ruptured towards the array and then decelerated. This rupture “jump” strongly focussed both the P-waves, producing the remarkable vertical PGA’s, and the S-waves, producing a strong pulse that dominated the velocity records (Archuleta, 1984). Thus, the model of Anderson and Luco (1983), which has a constant rupture velocity and constant slip, is inappropriate for this earthquake: there is relatively little slip on the fault near the array. Moreover, Archuleta (1984) locates the slip in the earthquake between 12 and 5 km so that the intermediate and near-field terms, which fall off as \( r^{-3} \) and \( r^{-2} \), respectively, should have little amplitude at the surface.

Second, the Imperial Valley is an extremely young sedimentary basin, and the near-
surface wave velocities are commensurately slow. Archuleta (1984) uses a shear wave velocity of $\beta = 400 \text{ m/s}$ and a density of $\rho = 1.8 \text{ gm/cm}^3$ for the upper 0.4 km of sediment. Thus, the radiated S-waves should be strongly amplified as they propagate through this velocity structure, although the impedance contrast argues for an average amplification of about 3.3 rather than 12 or 15. We note that the shape of the synthetic waveforms would also be distorted by this velocity structure and amplification. The fit of the synthetic transverse pulse to the “shape” of pulses recorded at stations 6 and 7, shown in Figure 8, must be regarded as fortuitous.

As a second comparative test of the Luco and Anderson (1983) model, we derive synthetics for the records written by the 1992 Landers, California, earthquake at the Lucerne Valley Power Station. The SMA-2 accelerograph was sited on relatively hard rock, approximately 2 km southwest from the Emerson fault segment of the Landers surface rupture. As a model for the rupture process we will use the rupture characteristics from the inversions performed by Wald and Heaton (1994). We will also use the wave velocities ($\alpha = 5.5 \text{ km/s}$ and $\beta = 3.14 \text{ km/s}$) in the second layer of their velocity model (extending from 1.5 to 4 km in depth) as the velocities of the half-space of the Luco and Anderson (1983) model.

In the Wald and Heaton (1994) inversions, the rupture extends 5-8 km below the surface on the Emerson Lake/Camp rock fault; the local rupture velocity was 2.1 km/s and the rupture front was inclined slightly so that the rupture at the surface lagged the rupture at depth. We note that Wald and Heaton (1994) fit both the displacement and velocity components obtained at Lucerne Valley so that their rupture model is already strongly constrained by the data that we will fit. Our purpose in this comparison is to demonstrate how much of the near-fault ground motions are contained in the Luco and Anderson model of fault fling.

At this writing, the directions of the horizontal components of the accelerograph are unspecified. The polarization of velocity on the “transverse” component, which is approximately three times the velocity on the “longitudinal” component, indicates that this component of motion is aligned nearly transverse to the local fault trace rather than east-west, which is the normal field deployment. The fits shown in Figure 9 then compare the “transverse” and “longitudinal” components of velocity and displacement with the synthetic motions transverse and parallel to the fault trace. We note that the synthetics determined by Wald and Heaton (1994) show the same polarization.

The synthetic waveforms plotted in Figure 9 are only sensitive to the horizontal rupture velocity $c_1$. Cursory inspection of equation (1) shows that the time enters the equation as $c_1 t$: the faster the rupture velocity, the more compressed the time function. The relatively slow rupture velocity $c_1 = 2.1 \text{ km/s}$ leads to a relatively wide time function. The velocity $c_2$ controls the inclination of the rupture front and exerts
a secondary effect on the pulse shapes. We have chosen $c_2 = -7.0 \text{ km/s}$ to match the inclination of the rupture front in the Wald and Heaton model.

The fit shown in Figure 9 is remarkable in that there are no free parameters. Essentially, we have used the parameters of Wald and Heaton (1994) in a uniform faulting model. The width of the longitudinal velocity pulse is somewhat underestimated by the synthetics, but this component is much weaker in amplitude and strongly affected by any rotations. Figure 9 shows that, for this station, the coherent faulting model fits almost all the velocity and displacement component in the transverse direction. As discussed before, the fit is predicated on the inversion results of Wald and Heaton (1994): clearly, the character of the rupture process near the Lucerne Valley station are strongly conditioned by their fit. What is surprising is that the transverse pulse shape from the Luco and Anderson (1983) exactly fits the data, despite significant complexity of the rupture process as the rupture nears the site.

Discussion and Conclusions

The two examples discussed above yield diametrically opposed results. For the Imperial Valley earthquake, the Luco and Anderson (1983) model significantly underestimates the recorded velocity and displacement. For the Landers earthquake, the model exactly fits the transverse pulse shape and amplitude. It is probable that this fit is partly fortuitous: the average rupture velocity for the 6 seconds before the rupture reaches Lucerne Valley station is about 2.0 km/s. The rupture deceleration and acceleration which occurs at the stepover between the Homestead Valley and Emerson fault segments thus yield shear-wave pulses which cancel each other out, in the direction of rupture, leaving only the signal from a smoothly propagating rupture.

Even if this fit is fortuitous, however, the Luco and Anderson (1983) model provides a simple method of calculating the near-field effects associated with the passage of the rupture front. Such a simple tool has a clear predictive value, despite the conditions which may have produced this particular fit. The misfit of the Imperial Valley recordings, however, show the limitations of this model. In particular, it cannot fit motions at and beyond the ends of the fault. Moreover, we have not discussed another serious failing of the model, that is, its underestimate of peak accelerations. We will extend this discussion to that problem in a later draft of this paper.
References


Figure 1. 1906 intensity plotted as a function of distance from the San Andreas fault, reprinted from Borcherdt et al. (1975). The intensity scale is the 1906 intensity scale; approximate equivalences with the Modified Mercalli Intensity Scale are shown on the right of the graph.
Figure 2. Description of the fault model and coordinate system, reprinted from Anderson and Luco (1983). (a) Three dimensional view. The plane $z = 0$ is the free surface. The plane shaded area represents the vertical fault area which extends from $x = -\infty$ to $x = +\infty$. (b) Plan view of the plane which contains the fault. The shaded portion represents the area which has ruptured at time $t$. Slip on the fault is independent of the location within the shaded area and constrained to be horizontal.
Figure 3. Synthetic displacement time histories from the model of Luco and Anderson (1983). The rupture extends from 2.0 to 10.0 km in depth, in a half-space with $\alpha = 6.0$ and $\beta = \alpha / \sqrt{3}$. The horizontal rupture velocity is $c_1 = 3.184$ km/s, nearly equal to the Rayleigh wave velocity, and the slip dislocation is $\Delta_o = 100$ cm.
Figure 4. Synthetic velocity time histories from the model of Luco and Anderson (1983). The rupture model is the same as that for Figure 3.
Figure 5. Displacement pulse area for the SH component of the transverse pulse radiated by a range of fault models. All the faults start at $z_d = 10$ km depth: the upper limit of the faults ranges from $z_u = 4.0$ to 0.125 km. The rupture velocity ranges from $0.65\beta$ to $0.9\beta$. 

**Displacement Pulse Area,** $\Xi(z_d = 10, z_u, c_1)$
Figure 6. Variation of peak acceleration, velocity, and displacement as a function of
the distance normal to the fault for three rupture models, reprinted from Anderson and
Luco (1983). All three models have the same moment per unit length, and the same
rupture velocities, $c_1 = 3.0 \text{ km/s}$ and $c_2 = \infty$. The solid line corresponds to a rupture
with $z_u = 1 \text{ km}$, $z_d = 9 \text{ km}$, and $\Delta_o = 100 \text{ cm}$; the dashed line corresponds to a rupture
with $z_u = 2 \text{ km}$, $z_d = 10 \text{ km}$, and $\Delta_o = 100 \text{ cm}$, while the dash-dot line corresponds to
a rupture with $z_u = 2 \text{ km}$, $z_d = 18 \text{ km}$, and $\Delta_o = 50 \text{ cm}$.
Figure 7. Effect of the vertical rupture velocity $c_2$ on the synthetic velocity for a strike slip fault with $z_u = 2$ km, $z_d = 10$ km, $y = 2$ km, $c_1 = 3.0$ km, and $\Delta_o = 100$ cm, reprinted from Anderson and Luco (1983). For the case on the left, with $c_2$ negative, the rupture occurs first at the lower edge of the fault and proceeds upwards, while for the case on the right, with $c_2$ positive, the rupture occurs first on the upper edge of the fault and proceeds downwards.
Figure 8. Comparison of 10 s of ground velocity observed at stations 6 and 7 of the El Centro array during the 1979 Imperial Valley earthquake, reprinted from Anderson and Luco (1983). The components are oriented perpendicular to the fault (230°). Peak-to-peak amplitudes of each trace are given on the right. The synthetic is calculated with \( \alpha = 5.0 \text{ km/s}, \beta = \alpha/\sqrt{3}, z_u = 1 \text{ km}, z_d = 10 \text{ km}, y = 1 \text{ km}, c_1 = 2.0 \text{ km/s}, \Delta_o = 100 \text{ cm}, \) with a rise time of 1.5 s for the ramp source time function.
Figure 9. Comparison of 25 s of ground velocity observed at station LUC during the 1992 Landers earthquake. The data components are obtained by correcting, integrating, and high-pass filtering the two horizontal components of the accelerograms. The corner frequency for the high-pass filter is 0.05 Hz. The synthetic components are oriented parallel and transverse to the fault. The synthetics are calculated with $\alpha = 6.0 \text{ km/s}$, $\beta = 3.15 \text{ km/s}$, $z_\ast = .5 \text{ km}$, $z_d = 6 \text{ km}$, $y = 2 \text{ km}$, $c_1 = 2.1 \text{ km/s}$, $c_2 = -7.0 \text{ km/s}$, $\Delta_\circ = 500 \text{ cm}$, with a rise time of 4.0 s for a Kostrov source time function; they were high-pass filtered using a first order Butterworth filter with a corner at 0.05 Hz, run both ways over the synthetic traces. The synthetic displacement plotted for the parallel component in the lower left hand corner has been divided by two for the comparison.
NEW DEVELOPMENTS IN ESTIMATING SITE EFFECTS ON GROUND MOTION

AN INTEGRATED METHODOLOGY FOR ESTIMATES OF SITE-DEPENDENT RESPONSE SPECTRA, SEISMIC COEFFICIENTS FOR SITE-DEPENDENT BUILDING CODE PROVISIONS, AND PREDICTIVE GIS MAPS OF STRONG GROUND SHAKING

by

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Abstract

Recent geotechnical measurements and strong-motion data from the Loma Prieta earthquake constitute a new empirical basis to account for local geological conditions in earthquake resistant design, site-dependent code provisions, and seismic hazard analyses. The geotechnical data afford new and simplified definitions of site classes. The strong-motion data yield empirical estimates of amplification factors for these new site classes and for specific sites in terms of mean shear-wave velocity. These recent results are used to suggest a new integrated methodology for estimates of site-dependent response spectra, site-dependent building code provisions, and predictive maps of strong ground shaking useful for both code provisions and the mapping of special study zones. These new developments in estimating site effects on ground motion are presented for review and comment by the earthquake engineering community.

This paper presents: 1) a set of step-by-step procedures for estimating site-dependent response spectra for design based on the new short- and mid-period amplification factors ($F_a$ and $F_v$), 2) a proposed update of site-dependent building code provisions (section 4.2.1 of the 1991 Recommended NEHRP provisions), and 3) a general framework for predictive GIS mapping of strong ground shaking for purposes building codes and special-study-zone policy development in conformance with California law AB-3897. Derivations for the new simplified site classes and their associated amplification factors as a function of mean shear velocity and input ground motion level are presented in appendices as justification for the methods presented.
Introduction

Recent earthquakes, especially those affecting Mexico City, Mexico, Leninakan, Armenia, and the San Francisco Bay region, California have reemphasized the important influence of local geologic deposits on the amounts of damage and resultant loss of life. In general, damage and loss of life in each of these earthquakes was concentrated in areas underlain by soft-soil deposits. These concentrations of damage have emphasized the need to modify code provisions to better account for the amplification effects of local geologic deposits.

The strong-motion recordings of the Loma Prieta earthquake constitute an important new data set for quantifying the response of local geologic deposits. They are one of the most extensive sets of in-situ measurements of amplification. The data were obtained at damaging levels of motion on a variety of geologic deposits in close proximity, ranging from very soft clays to hard rock. The recording sites were located at distances and azimuths so that influences of source characteristics and wave propagation were minimal or could be isolated in analyses of the data from those of local geologic deposits. The data were recorded in a region for which a large amount of previous geologic, geotechnical, and seismic data existed for use in understanding the results.

Extensive sets of borehole geologic, geotechnical and shear-wave velocity data collected since the definition of soil-profile types S1 - S4 form an important new basis upon which to improve definitions of site classes for design provisions. These data sets have been collected and analyzed in San Francisco, Los Angeles, Salt Lake City, Seattle and other urbanized regions of the United States. They have established important correlations between seismic response and mappable physical properties of various geologic units. These results provide an important new basis for improving procedures to account for the amplification effects of local geologic deposits in design and hazard mitigation procedures.

The Loma Prieta strong-motion data, borehole geotechnical data, and numerical modelling results provide a new basis to account for local geological conditions in earthquake resistant design codes. They provide unambiguous definitions of site classes and rigorous empirical estimates of site-dependent amplification factors. These new results are used herein to suggest an integrated methodology for: 1) estimates of site-dependent response spectra for design, 2) estimates of seismic coefficients as an update to section 4.2.1 of the Recommended NEHRP provisions, 3) predictive GIS mapping of strong ground shaking for code purposes and mapping of special study zones (CA law AB-3897).

The methodology for estimates of site-dependent response spectra is summarized as a step-by-step procedure with alternate techniques presented for each step. Subsequent commentary facilitates evaluation and selection of techniques appropriate for desired application. Data analyses presented in the appendices provides the justification for the methodology. These results are from Borcherdt (1994b).

The estimates of seismic coefficients, presented in the context of a proposed update to section 4.2.1, use the procedures for estimating site-dependent response spectra. These procedures are based on the new short- and mid-period amplification factors (Fa and Fv), which can be specified either from the values for the simplified site classes or as a function of mean shear-wave velocity. These results are modified from Borcherdt (1994a).

The methodology for predictive GIS mapping of strong ground shaking is based on the definitions of the new simplified site classes and their corresponding amplification factors. It provides a framework for preparation of predictive ground shaking maps, which account for the amplification effects of local geologic deposits and is useful for both code purposes and the mapping of special study zones. The methodology for mapping special study zones is presented in the context established for mapping liquefaction and landsliding potential; namely, Capability (Susceptibility) + Opportunity = Potential. The results are from Borcherdt, and others (1991) and Wentworth, and others (1991).
Methods for Estimating Site-Dependent Response Spectra

Definitions and step-by-step procedures for estimating site-dependent response spectra are given below. Justification for the methods is presented in appendices I and II.

Definition of free-field, site-specific response spectra, $S_A$

Free-field, site-specific response spectra with 5% damping ($S_A$) are defined as:

$$S_A = \text{Minimum for each period (T) of} \left\{ \begin{array}{l} I_a F_a \\ I_v F_v (1/T)^x \end{array} \right.$$  

where $I_a$ and $I_v$ are input ground motion levels for the short-period (acceleration) and mid-period (velocity) bands respectively, for an implied uniform ground condition.

$F_a$ and $F_v$ are average short- and mid-period amplification factors with respect to the uniform ground condition used for determination of $I_a$ and $I_v$.

$T$ represents period in seconds, and $x$ is the spectral decay exponent for the mid to long period band.

Parameters in equation 1 needed to estimate site-dependent response spectra are illustrated in Figure (1). Steps required to estimate the spectra are given below. Alternate techniques are provided for each step to facilitate estimates for design and code update purposes.

Step 1 Determine input ground motion levels, $I_a$ and $I_v$

Input ground motion levels for the short- ($I_a$) and mid- ($I_v$) periods bands, may be determined from either:

a) maps showing effective peak ground motion values, $A_a$ and $A_v$ (Algermissen et al., 1982), where input ground motion levels are specified as $I_a = 2.5 \times A_a$ and $I_v = 1.2 \times A_v$ with the implied uniform ground condition being site class "firm to hard rock" (SC-Ib) and $x = 2/3$,

or

b) maps showing spectral ordinates $S(0.3)$ and $S(1.0)$ (Algermissen, et al., 1991), where input ground motion levels for the 0.3 and 1.0 second periods may be specified as either,

i) $I_a = S(0.3)$ and $I_v = S(1.0)$, with $x = 1$ and the implied uniform ground condition being S2, which corresponds to a combination of site classes SC-II and SC-III designated here as SC-(II+III),

or
Step 2 Characterize local site conditions.

Local site conditions may be characterized in terms of mean shear velocity to a depth of 30 m (100 ft) by either:

a) site classification based on physical descriptions of the near surface materials as specified in Table 1,

or

b) inferred mean shear-wave velocity, using information on thickness and physical properties to infer shear velocities for each of the underlying layers based on established correlations as shown in Figures 2, and 3,

or

c) measured mean shear-wave velocity, using shear-wave travel time measured to a depth of 30 m (100 ft) below the surface,

Step 3 Determine site-dependent amplification factors, $F_a$ and $F_v$.

Site-dependent amplification factors for various input ground-motion levels $I_a$ and $I_v$ specified with respect to a particular uniform ground condition may be determined by either:

a) site classification (step 2a) with corresponding amplification factor for appropriate uniform ground condition tabulated in Table 2a or 2b,

or

b) mean shear velocity estimate (step 2b or 2c) with corresponding amplification factor for appropriate uniform ground condition plotted in Figures 4 and 5.

Amplification factors tabulated in tables 2a and 2b and plotted in Figures 4 and 5 are predicted as a function of mean shear-wave velocity $v$ for various input ground motion levels $I_a$ specified with respect to a uniform ground condition by the following equations:

$$F_a(v, I_a) = \left(\frac{v_0}{v}\right)^{m_a}, \quad (2a)$$

and

$$F_v(v, I_a) = \left(\frac{v_0}{v}\right)^{m_v} \quad (2b)$$
where

\[ m_a = \frac{\log(F_a(v_{SC,IV}, I_a))}{\log(v_0/v_{SC,IV})} \]  

(2c)

\[ m_v = \frac{\log(F_v(v_{SC, IV}, I_a))}{\log(v_0/v_{SC, IV})} \]  

(2d)

\[ v_0 \] is mean shear-wave velocity for the site class corresponding to uniform ground condition used to specify input ground-motion levels \( I_a \) and \( I_v \).

\[ v_{SC, IV} \] is mean shear-wave velocity for the "soft-soil" site class (SC-IV), and

\[ F_a(v_{SC, IV}, I_a) \] and \( F_v(v_{SC, IV}, I_a) \) are short- and mid-period amplification factors respectively, for the "Soft-soil" site class at input ground motion level \( I_a \) specified with respect to uniform ground condition firm to hard rock (SC-Ib) in Table 2a and with respect to S2 or combined site class SC-(II+III) in Table 2b.

Step 4 Calculate free-field, site-dependent, response spectra, \( S_A \).

Site dependent response spectra, as defined in equation 1, are calculated using input ground motion levels derived in step 1, mean shear velocity estimates inferred in step 2, and the amplification factors derived in step 3.

Comments on Steps 1-4

The procedures specified in steps 1-4 constitute a flexible and general methodology for estimating free-field, site-dependent, response spectra. The methodology affords the flexibility of selecting different techniques for estimating both input ground motion levels and amplification factors, depending on specific site requirements and available information. The methodology forms a general framework to incorporate new information and new procedures as they become available. The framework permits evaluation of various techniques for possible inclusion in an update of the code.

Estimation of Input Ground Motion Levels (step 1) Three options are suggested for inferring short- (\( I_a \)) and mid- (\( I_v \)) period input ground motion levels (step1). The first two options use published maps depicting anticipated ground-motion levels. The third option is suggested in order to permit ground motion estimates that might be derived from a number of existing as well as possible future empirical attenuation relationships or a variety of ground-motion prediction models. The wide range in materials included in site class S2 and ongoing revisions in spectral attenuation relations currently argue that option 1 is preferred for estimating input ground motion levels based on published maps.

The value for the spectral decay exponent "\( x \)" is under review. The value currently used is 2/3. The value being suggested for incorporation into the code is 1. For consistency with present convention we shall choose \( x = 2/3 \), if input ground motions are specified from the effective peak motion maps (step 1a) and \( x = 1 \), if they are specified using either steps 1b or 1c.
Selection of one of the options for estimates of input ground motion levels implicitly implies selection of
an implied uniform ground condition with respect to which subsequent amplification factors must be ex-
pressed. As an alternate procedure (step 1bii), input ground motion levels may be converted to another uni-
form ground condition by normalizing the input levels to another uniform ground condition using average
amplification factors for appropriate site classes. For example, input ground motion levels determined from
the spectral ordinate maps may be converted to input levels for firm to hard rock (SC-Ib; step 1bii), by
normalizing \( I_a \) and \( I_y \) by the amplification factors for site class SC-(II+III) with respect to SC-Ib, that is by
\( F_{a}^{SC-(II+III), I_{a}} \) and \( F_{v}^{SC-(II+III), I_{a}} \), respectively. For sites with mean shear velocities equal to the mean of
one of the site classes the two procedures yield identical results.

Characterization of Local Site Conditions (step 2) Three options are suggested for characterizing the
local site conditions in terms of mean shear wave velocity for purposes of estimating ground response or
amplification factors. The first option allows the site conditions to be characterized by classification of the
site using a description of the physical properties of the near surface materials. This classification in turn
allows the mean shear-wave velocity for the corresponding simplified site class to be assigned to the site. The
second and third options afford more quantitative characterizations in terms of either inferred or measured
estimates of mean shear velocity to a depth of 30 m (100 ft). These estimates of shear velocity permit more
accurate classification of the sites using the shear-wave velocity criteria specified in Table 1. They also permit
estimates of amplification as a continuous function of shear velocity using equations 2a-2d. Choice of tech-
nique depends on information available and intended site usage.

The physical property classification (Step 2a) of a site using the criteria specified in Table 1 may be
based only on a simple physical description of the near-surface materials. As such, this option affords a
simple and straightforward procedure for classifying the site and in turn assigning a mean shear-wave ve-
cocity. It is a procedure that should allow most sites to be readily classified based primarily on near-surface
information easily acquired at the site. As the amplification characteristics of geologic deposits tend to de-
crease with depth, a classification of a site based only on the near surface materials will tend to over estimate
the amplification characteristics of the site. Site classifications may be improved with additional information
on the physical properties and distribution of materials at depth. Any uncertainties in classifications based on
physical property descriptions can be resolved by assignment of the more conservative site class or further
studies to either infer or measure mean shear velocity to 30 m (100 ft).

Estimates of mean shear-wave velocity (Steps 2b and 2c) afford improved classifications of a site.
They may be inferred (step 2b) or measured directly (step 2c). Mean shear-wave velocity to a depth of 30 m
(100 ft), may be inferred using information on thickness and physical properties of the underlying layers. If
this information is available from borehole logs or other sources, then shear-wave travel-times and corre-
sponding velocities for each layer may be inferred from velocity measurements derived in other boreholes with
similar materials at similar depths. At most sites this technique should yield estimates of shear-wave velocity
more accurate than those inferred in step 2a. Information on other geotechnical parameters such as standard
penetration resistance, undrained shear strength, or void ratio also can be used to further refine estimates of
shear velocity (e.g., Fumal and Tinsley, 1985, pp 134-135).

Such inferences, based on the correlations between shear-wave velocity and physical properties used
for the definitions of the simplified site classes (Figures 2 and 3), permit relatively reliable inferences of mean
shear-wave velocity to a depth of 30 m (100 ft) at a site (Fumal, 1985, 1991). The data permit inferences of
interval velocities and, in turn, travel times for each depth interval. These interval travel times can then be
summed to determine the total travel time to 30 m (100 ft) and, in turn, the desired estimate of mean shear-
wave velocity. Similar correlations for other materials in other regions can be established readily using a
limited number of borehole logs for principal geologic units in the region. The correlations provide a frame-
work to which additional data may be easily added as it is collected.
The third technique for characterizing the local site conditions (step 2c) is direct measurement of mean shear-wave velocity using shear-wave travel times through the top 30 m (100 ft) of material. Procedures for such measurements in boreholes, as provided here for the San Francisco and Los Angeles regions (Figures 2 and 3) are described (e.g. Gibbs et al., 1975, 1976, 1977). This technique provides the most accurate characterization of a site for purposes of estimating amplification factors. It permits a site to be classified unambiguously. It permits quantitative estimates of amplification using either equations derived from the Loma Prieta strong-motion data or results derived from numerical models. Direct measurements of shear-wave velocity should be needed primarily for special projects.

**Estimation of Site Dependent Amplification Factors, $F_a$ and $F_v$ (step 3)** Quantitative estimates of site-specific amplification factors $F_a$ and $F_v$ are implied by the mean shear-wave velocity estimates used to characterize the local site conditions in step 2. Two options are suggested in step 3 for determining these factors with respect to a specified uniform ground condition for a given input ground motion level.

The first option (step 3a) suggests that the amplification factors be derived as a discrete function of site characteristics based on classification of the site into one of the four major site classes or associated subclasses (step 3a). The corresponding discrete amplification values are given in Table 2a, if the uniform ground condition is "Firm to hard rock", (SC-Ib) and in Table 2b if it is chosen as the combined site class (SC-(II+III)).

The second option (step 3b) suggests that the amplification factors may be estimated as a continuous function of shear-wave velocity using equation 2 or corresponding plots in Figures 4 and 5. Additional parameters needed to evaluate equations $2a$-$2d$ are given in Tables 2a and 2b. This approach yields more accurate estimates of amplification factors than might be derived from discrete estimates for the simplified site classes. This option is useful for sites that can not be readily classified or for sites with projects which warrant special study.
Site-Dependent Building Code Provisions  
(A Proposed Update of NEHRP Recommended Provisions - Section 4.2.1)

This section includes a proposed update to section 4.2.1 of the NEHRP recommended code provisions. The proposed revisions incorporate the short- and mid-period amplification factors $F_a$ and $F_v$, which can be specified from either the values for the new simplified site classes or the values implied by the mean shear-wave velocity at the site. The seismic coefficient $C_S$ computed in the following provision corresponds to the site-dependent response spectra, $S_A$, normalized by the building response modification factor, $R$, that is $C_S = S_A / R$. $S_A$ is computed with respect to input ground motion levels and amplification factors determined with respect to "firm to hard rock", SC-Ib as specified in Steps 1a, 2, 3, and 4. The proposed update of section 4.2.1 is shown below with additions indicated in bold lettering and deletions with strike-through. Table numbers in italics refer to tables in NEHRP provisions (from Borcherdt, 1994).

4.2.1 CALCULATION OF SEISMIC COEFFICIENT: When the fundamental period of the building is computed, the seismic design coefficient ($C_S$) shall be determined in accordance with the following equations:

$$C_S = 1.2 A_v F_v S / (R T^{20})$$

where:

$A_v =$ the coefficient representing effective peak velocity-related acceleration from Sec. 1.4.1 determined with respect to sites on "firm to hard rock" (SC-Ib),

$F_v =$ the mid-period amplification factor specified for various levels of $A_v$ either by Table 2a for the simplified site classes or by equations 2a through 2d as plotted in Figure 4b.

$S =$ the coefficient for the soil profile characteristics of the site in Table 3.2,

$R =$ the response modification factor in Table 3.3, and

$T =$ the fundamental period of the building determined in Sec. 4.2.2.

A soil-structure interaction reduction is permitted when determined using the "Appendix to Chapter 6", or other generally accepted procedures approved by the regulatory agency.

Alternatively, the seismic design coefficient ($C_S$) need not be greater than the following equation:

$$C_S = 2.5 A_s F_s / R,$$

where:

$A_s =$ the seismic coefficient representing the effective peak acceleration as determined in Sec. 1.4.1 with respect to sites on "firm to hard rock", SC-Ib,

$F_s =$ the short-period amplification factor specified for various levels of $A_s$ either by Table 2a for the simplified site classes or by equations 2a through 2d plotted in Figure 4a,

$R =$ the response modification factor in Table 3.3.
Predictive GIS Mapping of Strong-Ground Shaking

Predictive maps of strong ground shaking, which incorporate the amplification effects of local near surface deposits, can facilitate the implementation of site-dependent building code provisions, the mapping of special study zones for strong ground shaking, liquefaction and landsliding as specified by California Law AB-3897, and the improvement of regional seismic hazard analyses. Such predictive maps must necessarily be comprised of ground motions predictions for a uniform ground condition such as "firm to hard rock" plus predictions of the amplification effects of any near surface deposits. The simplified site classes and their associated amplification factors as specified in Tables 1 and 2 provide a rigorous basis for predictive ground motion maps, which account for the amplification effects of local geologic deposits. They suggest a methodology for predictive mapping, which integrates the procedures for estimates of site-dependent response spectra and the proposed update to section 4.2.1 of the NEHRP recommended code provisions.

Modern Geographic Information Systems (GIS) afford important new opportunities to compile, archive and modify extensive spatial data bases for purposes of predictive mapping. In particular, GIS technology provides powerful new tools to archive data pertaining to earthquake faults, distribution and character of geologic materials, topography, and water-table depth. These data, needed to map potential variations in ground shaking, liquefaction, landsliding, and water inundation, when incorporated with cultural information using GIS permit efficient assessment of Potential Earth Science Hazards (PESH) associated with earthquakes. Applications of GIS (ARC/INFO) to the archival of earth science data and the predictive mapping of strong ground shaking and liquefaction are discussed in detail by Wentworth and others (1991) and Borcherdt and others (1991). Their results are used here to illustrate methods for predictive maps consistent with the procedures for site-specific estimates of response spectra and the proposed code update presented in preceding sections.

Predictive GIS Mapping of Ground Motion Amplification

Mapping potential geographic variations in ground shaking in a region requires data available throughout the region. In most regions, data most readily available is geologic data presented in the form of two-dimensional geologic maps. Detailed information on the three-dimensional distribution of materials is generally not available throughout a region in sufficient detail to be incorporated into maps showing variations in ground response. Consequently, the practical production of predictive maps must necessarily be based on the distribution of geologic materials as mapped at the surface. Interpretations of the resultant maps should account for these limitations.

The physical properties of the simplified site classes defined in Table 1 and specified in Figures 2 and 3 permit the classification of mapped units into one of four major classes. These classes together with their corresponding amplification factors $F_a$ and $F_v$ as specified in Table 2 provide the basis for the preparation of predictive ground motion maps, which account in a general way for the amplification effects of local geologic deposits.

Correlations between measured amplifications, observed earthquake intensities, shear-wave velocities, and physical properties of geologic materials as mapped at the surface were used to establish four major site classes with distinct seismic response characteristics (Borcherdt and others, 1991). These four classes (see Table I, Borcherdt and others, 1991) correspond with minor refinements in notation to the simplified site classes defined in Table 1. Amplification capabilities ascribed to the four major classes as summarized in...
Table 4, are High to very high for “Soft soils” (SC-IV), Intermediate to high for “Stiff clays and sandy soils” (SC-III), Low to intermediate for “Gravelly soils and soft rock”, and Low to very low for “Firm and hard rock” (SC-I). Short- and mid-period amplification factors corresponding to these ascribed amplification capabilities for each site class are given as a function of input ground motion level with respect to “Firm to hard rock” (SC-Ib) in Table 2a and with respect to the combined site class (SC-II + SC-III) in Table 2b. These amplification factors associated with each classification used for mapping are the same as those used to develop estimates of site-dependent response spectra and seismic coefficients for the proposed code revision.

Examples of amplification capability maps developed for purposes of mapping special study zones are reproduced for two areas in the San Francisco Bay region in Figure 12 (Borcherdt, and others, 1991). These maps were prepared using physical property attributes as compiled using a GIS data base for some 38 units in the San Francisco Bay region (Wentworth and others, 1991, 1994). The maps shown in Figure 12 identify zones according to their generalized capability to amplify ground motion as identified from zones distinguished according to the distribution of surface materials. The maps are intended to ascribe general levels of amplification capability. The maps are not intended to predict actual ground motion amplification at specific sites, but instead to indicate areas that if underlain by sufficient thicknesses of mapped surface materials are likely to experience amplifications commensurate with those specified in Table 2. In general, softer materials overlie harder materials. Consequently, if thin layers of surface materials less than the minimum thickness specified in Table 1 exist in an area, the maps may tend to overestimate the amplification capability by one unit and thus provide incentive for additional site investigations depending on the nature of the project.

Strong correlations between the measured amplification factors $F_v$ as derived from the Loma Prieta strong motion data, weak-motion data, and observed intensity increments for both the 1989 and 1906 earthquakes in the San Francisco Bay region imply that intensity increments also can be ascribed to the four major site classes (Borcherdt, and others 1975, 1991). These correlations suggest that amplification capability maps as defined here serve to identify areas for which ground motion amplification may enhance input ground motions to levels sufficient to significantly damage vulnerable structures. The maps identify areas for which special studies may be needed to develop more precise ground motion predictions, based on additional site-specific information. Amplification capability maps are similar in concept to liquefaction susceptibility maps (Youd and Perkins, 1978) in that both maps are useful for identifying areas for which the specific phenomena may be a problem.

**Predictive GIS Mapping of Ground Shaking on a Uniform Ground Condition**

(Ground Shaking Opportunity Maps)

The opportunity for strong ground shaking to exceed some specified level for sites on a uniform ground condition, such as bedrock depends on the size, location, crustal attenuation characteristics, and frequency of occurrence of earthquakes. A variety of procedures exist to account for these factors in the form of predictive probabilistic ground-motion maps for a uniform ground condition (Algermissen, and others, 1982; 1991). Such maps often depict contour levels that selected ground motion parameters will exceed with a specified return period. To maintain consistency with liquefaction mapping terminology (Youd and Perkins, 1978) these probabilistic maps have been referred to as opportunity maps for an exceedence in ground shaking or, for brevity, exceedence opportunity maps (Borcherdt, and others, 1991). Such maps provide the basis for inference of input ground motion levels $I_x$ and $I_y$ for a uniform ground condition as described in Step 1.

An example of a predictive map developed for purposes of mapping special study zones is reproduced for two areas of the San Francisco peninsula in Figure 13 (Borcherdt, and others, 1991). This map depicts contours for earthquake intensity for a repeat of the 1906 earthquake on the San Andreas fault for the uniform ground condition “Firm to hard rock”, (SC-Ib). The contours were predicted using the attenuation relation...
derived from 1906 intensity data for sites underlain by rocks of the Franciscan Complex \( \text{Intensity} = 2.69 - 1.90 \log \text{[distance (km)]} \); 1906 S.F. scale, Borcherdt, and others, 1975). Specification of the contours in terms of a selected return period permits the map to be interpreted in terms of the probability (opportunity) for intensity to exceed specified levels. Similar maps also can be produced for a uniform ground condition using a variety of attenuation and scaling relations derived for other ground motion parameters such as peak acceleration, peak velocity or pseudo-spectral response parameters. Such maps can be readily generated with GIS.

**Predictive GIS Mapping of Ground Shaking for Non-Uniform Ground Conditions**

(Ground Shaking Potential Maps)

The potential for strong ground shaking to exceed a specified level depends on the opportunity for input motions on a standard ground condition to exceed a particular level and the capability of local deposits to amplify incoming motions. Consequently, maps showing the potential for an exceedent in strong motion represent the composite or superposition of maps showing exceedent opportunity and amplification capability. Such predictive maps for ground motion parameters such as peak acceleration, peak velocity, and pseudo-spectral response for the short- and mid-period bands together with the amplification factors \( F_a \) and \( F_v \) associated with each of the simplified site classes provide a rigorous basis for preparation of such maps. These predictive maps of strong ground shaking, which account for variations in the local geology provide the basis for predictive maps useful for implementation of building code provisions.

An example of maps showing potential variations in earthquake intensity for two areas in San Francisco are reproduced in Figure 14 (Borcherdt, and others, 1991). The map was constructed as composite of the intensity map for a uniform ground condition ("Firm to hard rock", Figure 13) and the amplification capability map (Figure 12). Incorporation of information on the probability of occurrence permits the map to be interpreted in terms of the potential for an exceedent in earthquake intensity for an earthquake similar in size and location to the 1906 earthquake on the San Andreas fault. In order to maintain consistency with liquefaction terminology (Youd and Perkins, 1978), these maps may be designated as potential maps for an exceedent in ground shaking or for brevity exceedent potential maps for ground shaking or ground shaking potential maps.

**Predictive GIS Mapping of Special Study Zones for Ground Shaking**

Legislation for the state of California (Seismic Hazards Mapping, Act AB-3897, 1990) mandates that guidelines and priorities be established for preparation and utilization of maps delineating special study zones for strong ground shaking, liquefaction, and landsliding. Such maps and corresponding policies for interpretation are an important new mechanism for implementation of improved hazard mitigation measures. Such maps developed from ground shaking potential maps that could be adopted as an update to the code would facilitate development of consistent state and national hazard mitigation policies.

Predictive maps of strong ground shaking which account for variations in the local geologic deposits readily lend themselves to the definition of special study zones. A simple criteria for defining special study zones is the selection of some critical level of ground shaking with areas above this level designated as special study zones. Such special study zone maps, based on the simplified site classes (Table 1) and peak ground motion parameters or short- and mid-period spectral response parameters permit straightforward interpretation in terms of proposed revisions of the building codes. Maps prepared on the basis of simplified site classes and earthquake intensity facilitate interpretation for purposes of land-use and pre- and post-disaster planning.

Examples of special study zone maps are reproduced for two areas of the San Francisco Bay region in Figure 15 (Borcherdt, and others, 1991). This predictive GIS map was prepared from the potential map for...
earthquake intensity (Figure 14). The map was derived by specifying a critical level for intensity at the midpoint between levels of very strong and violent on the 1906 intensity scale.

Maps showing special study zones define most concern for strong ground shaking (Figure 14). The maps indicate that zones near the causative earthquake source, zones underlain by "Soft soils" (SC-IV), and zones at intermediate distances with mapped surficial materials having some amplification capability, are special study zones. For many sites, additional investigation may reveal that sufficient thickness of soil does not exist for the site to be of concern. For other sites, special land-use or design practice may be indicated by further investigation. Clearly, for special study zone maps to be effective in mitigating earthquake hazards, policies regarding type of site investigation and mitigative measures must be developed and implemented. These policies should be integrated and consistent with those developed for purposes of site-dependent building codes.

**Conclusions**

Steps for estimating site-dependent response spectra are: 1) determine input ground-motion levels for short-period, $I_a$ and mid-period, $I_v$ bands, 2) characterize the local site conditions, 3) determine the corresponding amplification factors, $F_a$ and $F_v$, and 4) calculate free-field, site-dependent response spectra, $S_A$, defined as

$$S_A = \text{Minimum for each period (T) of } \{ I_a F_a, I_v F_v (1/T)^x \}$$

where $x$ is a chosen spectral decay exponent. Alternate techniques, described for each step, facilitate determination of the most appropriate procedures for purposes of estimating earthquake resistant design spectra and for revision of current code provisions.

Suggested designations, general physical descriptions, intervals for mean shear-wave velocity and ascribed amplification capabilities for the simplified site classes are:

**SC-I, Hard and firm rock;**
- **SC-Ia, Hard rock,** $[> 1400 \text{ m/s}, (> 4600 \text{ ft/s})]$,
- **SC-Ib, Firm to hard rock,** $[700-1400 \text{ m/s}, (2300-4600 \text{ ft/s})]$.

**SC-II, Gravelly soils and soft rock,** $[375-700 \text{ m/s}, (1230-2300 \text{ ft/s})]$, Low to intermediate

**SC-III, Stiff clays and sandy soils,** $[200-375 \text{ m/s}, (660-1230 \text{ ft/s})]$, Intermediate to high

**SC-IV, Soft soils,**
- **SC-IVa, Non-special study,** $< 37 \text{ m (120 ft) thick},$
- **SC-IVb, Special study,** $> 37 \text{ m (120 ft) thick}$.

These revised definitions of the site classes simplify and reduce ambiguity in the classification of sites for site-dependent design purposes and provide a rigorous basis for an update to site-dependent code provisions and predictive mapping of strong ground shaking, which takes into account effects of local geologic deposits.

The Loma Prieta strong-motion data show that both short-period $F_a$ and mid-period $F_v$ amplification factors are needed to describe the horizontal response characteristics of local geologic deposits. These amplification factors are described by the following equations:
\[ F_a = \left( \frac{\nu_0}{\nu} \right)^{m_a} \]

and

\[ F_v = \left( \frac{\nu_0}{\nu} \right)^{m_v}, \]

where

1) \( \nu \) is the mean shear-wave velocity to 30 m (100 ft) and may be either inferred from physical properties or measured directly at a site,
2) \( \nu_0 \) is the average shear-wave velocity for the site class chosen as the uniform ground condition, and
3) \( m_a \) and \( m_v \) are implied by the amplification factor for soft soil deposits specified at the 0.1g input ground motion level by the Loma Prieta strong-motion data and at higher levels by extrapolation using numerical modelling results.

These equations yield well-defined estimates of amplification at various input ground motion levels both as discrete functions of shear-wave velocity for the simplified site classes as well as continuous functions for sites with more detailed information. These equations, together with estimates of input ground-motion levels for the short and mid period bands provide a well-defined, quantitative framework for improved estimates of site-dependent response spectra, \( S_A \) and seismic coefficients \( C_S \) for consideration as an update to site-dependent code provisions.

The simplified site classes and their associated amplification factors as specified in Tables 1 and 2 provide a rigorous basis for predictive ground motion maps which account for the amplification effects of local geologic deposits. They suggest a methodology for predictive mapping, which integrates the procedures for estimates of site-dependent response spectra and the proposed update to section 4.2.1 of the NEHRP recommended code provisions. Modern Geographic Information Systems (GIS) provide a powerful new tool to compile, archive and modify extensive spatial data bases for purposes of predictive mapping for strong ground shaking to better accounts for the amplification effects of local geologic deposits.

**Acknowledgments**

The results regarding estimates of site-dependent response spectra and seismic coefficients for the proposed code update was stimulated by the authors participation as a member of a working committee to consider site response and revisions of the 1991 Recommended NEHRP provisions. A part of this manuscript was presented at the national workshop of similar title (G. Martin, ed., 1994). Interactions with other members of the committee (C.B. Crouse, R. Dobry, I.M. Idriss, W.B. Joyner, G.R. Martin, M.S. Power, E.E. Rinne, and R.B. Seed) helped shape many of the concepts presented here. Conversations and exchange of results with Professors R. Seed, R. Dobry, and M. Power were especially helpful. T. Fumal independently derived shear-wave velocity intervals for the simplified site classes in agreement with those shown here. The predictive GIS mapping results are based on the significant and ongoing efforts of C. Wentworth and T. Fitzgibbon of the USGS in developing an extensive GIS geologic data base for the San Francisco Bay region and the contributions of A. Janssen in cooperation with faculty A. Kiremidjian and K. Law of Stanford University.
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Appendix I

Empirical Justification For Simplified Site Classes

Extensive sets of geologic, geotechnical and seismic data collected within the last two decades yield new results for simplifying the definitions of site classes used for earthquake resistant design purposes. New correlations have been established between parameters known to characterize the response of near surface deposits such as shear velocity and parameters describing physical properties that can be mapped on a regional scale. These correlations, together with recent strong-motion amplification observations, allow mappable site classes to be defined with distinct seismic response characteristics that simplify and reduce ambiguity in the classification of sites for seismic design purposes.

Definitions of the simplified site classes are given in Table 1. These definitions are based on recent comprehensive sets of in-situ data collected in order to determine relationships between mappable properties of near surface materials, shear-wave velocity, and ground motion amplification (Borcherdt et al., 1978, Fumal, 1978; Fumal and Tinsley, 1985; Borcherdt, et al., 1991). These in-situ data derived from detailed borehole logs are published for about 130 sites in the San Francisco and Los Angeles regions (Fumal, 1978; Borcherdt et al., 1978; Gibbs et al., 1975; 1976; 1977; Gibbs et al., 1980; Fumal, et al. 1981, 1982, 1984, Fumal and Tinsley, 1985; Gibbs et al., 1992, and Fumal, 1991).

Shear-wave velocity and physical property correlations as published for about 130 sites in the San Francisco and Los Angeles regions are replotted in Figures 2 and 3, from Borcherdt, et al., 1978, Fumal, 1978, and Fumal and Tinsley, 1985. These figures show that the soil and rock units in the regions can be subdivided for mapping purposes into about thirteen groups distinguishable on the basis of shear velocity and mappable physical properties (Fumal, 1978; Borcherdt et al., 1978). Inspection shows that of these thirteen groups, four major classes can be distinguished on the basis of physical properties and mean shear-wave velocity. These four major classes are defined in Table 1. The three criteria specified to distinguish the four classes are physical properties, mean shear velocity to 30 m (100 ft) and minimum thickness. The minimum thickness criteria was selected to ensure that sufficient material is present for resonant amplification to occur in the period band of engineering interest (>0.1 s). These criteria have been demonstrated to define classes with distinguishable amplification characteristics (Borcherdt, et al., 1991) useful for mapping purposes. Roman numeral designations are suggested for the site classes (that is, SC-I through SC-IV) in order to distinguish them from site class designations S1 through S4 and seismic performance categories A through D currently used in code provisions (see e.g. Section 3.3 and 3.4 of the NEHRP recommended provisions).

The four main site classes (Table 1), as described in terms of simple physical properties, are SC-I, firm to hard rock; SC-II, soft to firm rock and gravelly soils; SC-III, stiff clay and sandy soils; and SC-IV, soft soils. Subdivisions of these major site classes are provided for SC-I and SC-IV. Site class I for rock is subdivided in order to distinguish sites underlain by very hard rock, (SC-Ia), for example some sites in the eastern United States, from more prevalent sites in the western United States underlain by firm to hard rock, (SC-Ib). Site class IV for soft soils is subdivided in order to distinguish soft soils less than 37 m (120 ft) thick that do not present special stability problems (SC-IVa) from those that do present special problems or are especially thick (SC-IVb). Subclass SC-IVb is defined on the basis of geotechnical studies, foundation investigations, and numerical modeling results (R. Dobry and R. Seed, pers. commun. 1992). Soft soils in subclass SC-IVb present special seismic stability problems that require special studies prior to site development and/or construction.

Comparison of these definitions with those for the original classes S1-S4 shows that in general, S1 corresponds to SC-I, S2 is included in but not equivalent to SC-II and SC-III, and S3 and S4 are included in
SC-IV. Definitions for the major site classes SC-I through SC-IV, independent of thickness except for a minimal thickness, will simplify and reduce ambiguity in the classification process.

The shear-wave velocity intervals, as specified for each site class in Table 1, are implied by the correlations between physical properties and shear-wave velocity summarized in Figures 2 and 3. Some flexibility exists in the choice of these endpoints, however, choice not based on empirical data can lead to significant discrepancies between physical properties and mean shear-wave velocity, especially for soft soils. For example, if the upper limit for seismic shear-wave velocity were restricted to 180 m/s (600 ft/s) for the soft soil site class, then a number of sites underlain by more than 3 m (10 ft) of bay mud in the San Francisco Bay region (e.g. the strong motion station at the San Francisco airport, Borcherdt and Glassmoyer, 1992) would be improperly classified as stiff clays or sandy soils. Similarly, if the range for shear-wave velocity for soft rock and gravelly soils was extended to 750 m/s (2460 ft/s), then a number of sites on oölite to hard rocks of the Franciscan Complex would be classified as soft rock. Consequently, definitions of the site classes must be consistent with correlations between amplification, mean shear-wave velocity to 30 m (100 ft), geotechnical parameters, and physical descriptions of the materials, otherwise inconsistent classifications of sites and biases in strong-motion attenuation relationships can result. Further simplification in the classification process might also be considered by reducing the number of classes. For example, site classes SC-II might be combined with SC-III. However, inspection of amplification curves (Borcherdt, et al., 1991) shows that the scatter in the data for such a combined class is sufficiently large to suggest no statistical difference between the amplification factors for the remaining site classes.

The criteria used to define the site classes in Table 1 are intended to be universally applicable. They are based on physical property descriptions used for standard mapping purposes consistent with other standard classification systems (e.g. Unified Soil Classification, U.S. Corps of Engineers). They are rigorously defined in terms of a large and published data set. They are intended to permit the unambiguous classification of essentially all sites ranging from the softest soils to the hardest rocks. Upon review of this manuscript M. Celebi (pers. commun.) pointed out that the shear-wave velocity intervals derived here for the site classes are nearly identical to intervals that had been previously developed for building codes in Turkey (Earthquake Research Institute, 1975). The only endpoint that is different from those in Table 1 is that separating SC-II from SC-III. It is 400 m/s in the Turkish code instead of 375 m/s as derived here from the seismic borehole data.

Extensive and well documented data sets such as that summarized in Figures 2 and 3 provide a rigorous basis on which to differentiate sites according to the simple criteria specified. The resulting site classes provide a rigorous basis for inferring special study zones for strong ground shaking in response to California Seismic Hazards Mapping Act (California Law AB-3897) as well as development of site-specific code provisions. The site classes as defined have been ascribed amplification capabilities ranging from Low to very low for SC-I to High to very high for SC-IV (Borcherdt and others, 1991). These amplification capabilities, ascribed on the basis of correlations between shear-wave velocity and measured strong-motion amplification, are consistent with updates subsequently provided herein.
### Table 1 — Site classes for site-dependent code provisions as derived from detailed geotechnical studies.

<table>
<thead>
<tr>
<th>SITE CLASS</th>
<th>NAME</th>
<th>(1) General Description</th>
<th>(2) Mean Shear-Wave Velocity</th>
<th>(3) Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>minimum ft/s</td>
<td>average ft/s</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>m/s</td>
<td>ft/s</td>
</tr>
<tr>
<td>SC-I</td>
<td>SC-Ia</td>
<td>FIRM and HARD ROCKS</td>
<td>4600</td>
<td>1400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HARD ROCKS</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>(e.g. metamorphic rocks with very widely spaced fractures).</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SC-Ib</td>
<td>FIRM to HARD ROCKS</td>
<td>2300</td>
<td>700</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e.g. granites, igneous rocks, conglomerates, sandstones, and shales with close to widely spaced fractures).</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SC-II</td>
<td>B</td>
<td>GRAVELLY SOILS and SOFT to FIRM ROCKS</td>
<td>1230</td>
<td>375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e.g. soft igneous sedimentary rocks, sandstones, and shales, gravels, and soils with &gt; 20% gravel).</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SC-III</td>
<td>C</td>
<td>STIFF CLAYS and SANDY SOILS</td>
<td>660</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e.g. loose to v. dense sands, silt loams and sandy clays, and medium stiff to hard clays and silty clays (N&gt;5 blows/ft)).</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SC-IV</td>
<td>D</td>
<td>SOFT SOILS</td>
<td>330</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>SC-IVa</td>
<td>NON-SPECIAL STUDY SOFT SOILS</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e.g. loose submerged fills and very soft to soft (N&lt;5 blows/ft) clays and silty clays &lt; 37 m (120 ft) thick).</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SC-IVb</td>
<td>SPECIAL STUDY SOILS SOFT SOILS* *</td>
<td>330</td>
<td>100</td>
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<td></td>
<td></td>
<td>(e.g. liquefiable soils, quick and highly sensitive clays, peats, highly organic clays, very high plasticity clays (Pl&gt;75%), and soft soils more than 37 m (120ft) thick).</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(*) Mean shear velocity to a depth of 30 m (100 ft). (***) Site-specific geotechnical investigations recommended for this class. (***) Designation initially suggested R. Seed (written commun., 1992).
Table 2a -- Short- and mid-period amplification factors* with respect to uniform ground condition "firm to hard rock", SC-Ib, for site classes defined in Table 1.

<table>
<thead>
<tr>
<th>Input Ground Motion on SC-Ib</th>
<th>Site Class (shear velocity, ( v ) (m/s))</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SC-la (Ao)</td>
<td>SC-Ib (A)</td>
</tr>
<tr>
<td>( l_g ) (g)</td>
<td>1620</td>
<td>1050</td>
</tr>
<tr>
<td>0.1</td>
<td>0.35</td>
<td>0.9</td>
</tr>
<tr>
<td>0.2</td>
<td>0.25</td>
<td>0.9</td>
</tr>
<tr>
<td>0.3</td>
<td>0.10</td>
<td>1.0</td>
</tr>
<tr>
<td>0.4</td>
<td>0.05</td>
<td>1.0</td>
</tr>
<tr>
<td>0.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Short-period Amplification Factors \( Fa \) with respect to "firm to hard rock" SC-Ib

<table>
<thead>
<tr>
<th>( l_g ) (g)</th>
<th>( m_a )</th>
<th>( m_v )</th>
<th>( m_t )</th>
<th>( m_w )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.35</td>
<td>0.8</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td>0.2</td>
<td>0.65</td>
<td>0.8</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td>0.3</td>
<td>0.60</td>
<td>0.8</td>
<td>1.0</td>
<td>1.4</td>
</tr>
<tr>
<td>0.4</td>
<td>0.53</td>
<td>0.8</td>
<td>1.0</td>
<td>1.4</td>
</tr>
<tr>
<td>0.5</td>
<td>0.45</td>
<td>0.8</td>
<td>1.0</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Mid-period Amplification Factors \( F_v \) with respect to "firm to hard rock" SC-Ib

* Amplification factors are predicted by \( Fa = (1050/v)^*m_a \) and \( F_v = (1050/v)^*m_v \) (see text, equation 2).

** \( F_a \) and \( F_v \) factors for SC-IV are inferred at the 0.1g level from the Loma Prieta strong motion data and for higher levels from numerical modelling results (R. Seed and R. Dobry, pers. commun., 1992).
\[ v = \log[F_v(v_{SC-IV}, I_v)] / 0.845 \]  
(4d)

where the amplification factors for the soft-soil site class \( F_a(v_{SC-IV}, I_a) \) and \( F_v(v_{SC-IV}, I_v) \) are given at the 0.1g level by equations 3a and 3b evaluated at the midpoint of the shear-wave velocity interval (150 m/s) and at higher levels of motion by values specified for SC-IVa in Table 2a implied by numerical modelling results (Seed et al., 1992). Equations 4a and 4b together with values for \( m_a \) and \( m_v \) implied by 4c and 4d provide an especially simple basis for quantifying site response characteristics for purposes of earthquake resistant design and code provisions. Similar simple equations predicting amplification factors with respect to SC-(II+III) are

\[ F_a = (425 / v)^{m_a} \]  
(5a)

and

\[ F_v = (425 / v)^{m_v} \]  
(5b)

where, \( m_a \) and \( m_v \) are specified in Table 2b and readily inferred from 3a and 3b.

Parameters for the equations based on the simple extrapolation assumptions, (4a-4d) at the 0.1g level agree to within the first decimal place with those derived by regression analysis (4a - 4b). This agreement shows that the amplification factors derived from the simple extrapolation assumptions are well within the uncertainty indicated by the regression analysis. This agreement further suggests that the shear-velocity intervals for the site classes, the extrapolation equations, and the empirical regression curves as derived are consistent.

These expressions suggest that the average amplification at a site is proportional to the impedance ratio with respect to the uniform ground condition raised to an exponent, whose magnitude is the slope of the linear regression curves determined by the logarithms of the shear velocities and the amplification factors for the soft-soil site class (SC-IVa) and the chosen uniform ground condition at the specified input ground motion level.

The amplification factors predicted with respect to "Firm to hard" rock by equations 2 and 4 (Table 2a) are in good agreement with those derived by consensus based on agreement between Loma Prieta strong motion data (Borcherdt and Glassmoyer, 1992; 1993), numerical modeling results (Seed, et al., 1992), parametric studies (Dobry, et al., 1994), and subsequent modifications based on expert opinion (Martin, 1994). The amplification factors predicted with respect to "Firm to hard rock" (SC-Ib; Table 2a) agree exactly within the accuracy specified for most of the site classes. For those predictions that do differ the difference generally is less than 10 percent.

These general equations constitute a rigorous framework for estimating site-dependent spectra for earthquake resistant design. They permit various techniques to be compared for purposes of developing site-dependent code provisions. They afford simple and straightforward estimates of site-specific amplification factors as a continuous function of site characteristics, as well as discrete estimates for the various site classes. They yield amplification factors that can be readily refined in a self consistent fashion as additional empirical and theoretical results become available regarding the response of soft-soil deposits (SC-IV).
The linear form of the regression curve in Figure 6 suggests a simple and well-defined procedure for extrapolation. Specifically, it suggests that the strong-motion amplification factors derived from the Loma Prieta earthquake may be extrapolated based on two simple assumptions, namely:

1) the form of the regression curve relating amplification to mean shear-wave velocity remains the same at higher levels of motion, (i.e. the functional relation between the logarithms of amplification and mean shear velocity remains a straight line at higher levels of motion), and

2) the effects of nonlinearity on the chosen uniform ground condition are negligible.

These two simple assumptions imply that the straight lines for each level of input motion intersect the two points defined by the mean amplification and shear velocity of the soft-soil deposits (SC-IV) and the mean amplification and shear velocity of the chosen uniform ground condition. Consequently, as the amplification factors for the chosen uniform ground condition are necessarily unity, the extrapolation problem is completely determined by specification of the amplification factors at successively higher levels of motion for the soft-soil site class. For input ground motion levels near 0.1g these amplification levels can be specified by the empirical regression curves (equations 3a and 3b) for the Loma Prieta strong-motion data. For higher levels of motion they are inferable currently from laboratory and numerical modelling results (Seed, et al., 1992; R. Dobry, oral commun.) and eventually from in-situ data as other large earthquakes are recorded at higher levels of input motion. The resulting short-period ($F_a$) and mid-period ($F_v$) amplification factors as a function of mean shear velocity ($v$) and input ground motion level specified with respect to a particular uniform ground condition are given by equations 2a-2d. These equations provide a rigorous framework for extrapolation based on well-defined data and assumptions.

Amplification factors predicted by equation 2 for the simplified site classes (Table 1) are given in Tables 2a and 2b for uniform ground conditions corresponding to SC-Ib and SC-(II+III), respectively. Corresponding site-specific amplification factors predicted continuously as a function of mean shear velocity are plotted with linear scales in Figures 4 and 5. Equation 2 is replotted in Figure 8 with logarithmic scales. Plots of equation 2 with logarithmic scales readily illustrates the procedure for derivation of equation 2 as simple straight lines through the points determined by the logarithms of the amplification factors and mean shear velocities for the soft-soil site class and that of the chosen uniform ground condition.

Equations 2 and 3 can be written more simply if expressed only with respect to a single uniform ground condition. The resulting equations specified with respect to "Firm to hard rock", (SC-Ib) with mean shear velocity in m/s are

$$F_a = (1050 / v)^{m_a}$$

where, $m_a$ for various input ground motion levels are specified in Table 2a from the simple expression

$$m_a = \log[F_a(v_{SC-IV}, I_a)] / 0.845$$
\[ F_a = \left( \frac{997}{\nu} \right) ^{0.36} \]  
\[ F_v = \left( \frac{1067}{\nu} \right) ^{0.64} \]  

where \( \nu \) is the mean shear-wave velocity to 30 m (100 ft) and may be either inferred from physical properties or measured directly.

These empirical equations represent simple closed form expressions, useful for estimating site-specific amplification factors. The equations together with correlations between shear-wave velocity and physical properties (Fumal, 1978; Fumal and Tinsley, 1985) provide a rigorous estimate of amplification factors for sites and site classes based on physical property descriptions. These equations suggest the plausible result that the amplification factors are a function of the seismic impedance for the surficial material at the site with respect to "Firm to hard rock", (SC-Ib) raised to some power.

Amplification factors for the 0.1g input ground motion level as predicted by equations 3a and 3b are plotted both continuously and discretely for the simplified site classes in Figure 8. They are tabulated for the 0.1g level and the simplified site classes in Tables 2a and 2b. These empirical amplification factors are in good agreement with those derived independently based on numerical modelling of the Loma Prieta strong motion response (Seed, et al., 1992) and those derived based on parametric studies of several hundred soil profiles (R. Dobry, oral. commun., 1992). The Loma Prieta amplification factors at the 0.1g level provide a new empirical basis for estimates of site-dependent response spectra and possible code revisions (Borcherdt, 1994).

Amplification Factors Extrapolated From Loma Prieta Strong-Motion

The amplification factors implied by the Loma Prieta strong motion data were derived from input ground motion levels on firm to hard rock near 0.1g. For input ground-motion levels greater than this level, little or no empirical data exist on site response, especially for soft soils. Consequently, amplification estimates at these higher levels of motion must necessarily be based on laboratory and theoretical modelling considerations. An important issue in this regard concerns the extent to which the average amplification factors as inferred from in-situ strong-motion recordings extrapolate linearly to larger input ground motion levels.

In general, laboratory and theoretical results suggest that damping increases and shear modulus or velocity decreases for large strain levels as might occur near soil failure (Seed, et al., 1974; Idriss, 1990). These nonlinear characteristics of soil response can serve to increase impedance ratios and therefore increase amplification effects. They also can increase the intrinsic attenuation of the soil layer or the damping of seismic waves and therefore reduce the amplification effects. They can result in decreased shear modulus and seismic velocity and therefore lengthened fundamental response period. As strain levels in general increase with increasing softness of the soil deposits and increasing frequency of the wave, laboratory and theoretical results suggest these nonlinear effects should be more apparent for the softer soil deposits and for input ground motions in the shorter period or higher frequency range. Consequently, these nonlinear effects should be greater for \( F_a \) than \( F_v \) and increase with softness or decreasing shear velocity of the deposit. The strong-motion data from the Loma Prieta earthquake provide a new empirical basis for calibrating and extrapolation to higher levels of motion based on laboratory and numerical modelling results.
Appendix II
Empirical Justification For Site-Dependent Amplification Factors

The strong-motion recordings obtained from the Loma Prieta Earthquake of October 17, 1989 are an important new data set for characterizing the response of various geologic deposits to damaging levels of ground shaking. They provide quantitative measures of the in-situ response of various geologic deposits to damaging levels of shaking. Such observations obtained along a narrow range of azimuths on a variety of different types of geologic deposits in close proximity provide a new empirical basis from which to infer average amplification factors for construction of site-specific response spectra. As peak ground motion levels were near 0.1g for sites underlain by rock in the San Francisco Bay region, these amplification factors are especially relevant for input ground motions on similar rocks at levels less than or near these values. For larger input ground motion levels as might occur closer to the epicenter or for larger earthquakes, these empirical amplification factors constitute a base from which to extrapolate amplification factors using numerical modelling results. Derivation of the strong-motion amplification factors and techniques for their inference follow.

Amplification Factors Implied By The Loma Prieta Strong-motion Data

Average amplification factors for 35 free-field sites, as inferred from the strong-motion recordings of the Loma Prieta earthquake are given in Tables 3a and 3b. These amplification factors correspond to average spectral ratios for vertical and average horizontal ground motion (Borcherdt and Glassmoyer, 1992). The ratios summarized in these tables have been computed with respect to "local rock" sites with peak motions near 0.1g. Each of the ratios has been normalized by hypocentral distance and adjusted to the common ground condition "Firm to hard rock", (SC-Ib) of the Franciscan formation (KJf).

The spectral ratios represent averages over the short-period band (0.1-0.5 s), intermediate-period band (0.5-1.5 s), long-period band (1.5 -5.0 s), mid-period band (0.4-2.0 s), and entire-period band (0.1-5.0 s). Corresponding ratios of peak acceleration, velocity, and displacement and average spectral ratios for the individual radial and transverse components of motion are given elsewhere (Borcherdt and Glassmoyer, 1992: Borcherdt and Glassmoyer, 1993). Mean shear-wave velocity to a depth of 30 m (100 ft) as either measured or estimated for each site by Fumal (1992) are summarized in Tables 3a and 3b.

Regression curves for average horizontal spectral amplification as a function of mean shear velocity for the short-, intermediate-, long- and mid-period bands are provided in Figures 6a through 6d. 95% confidence limits for the observed values and for the ordinate to the true population regression line also are shown. These curves show that, in general, the average horizontal spectral amplification increases with decreasing mean shear velocity. Replotting the curves with linear scales (Figure 7) emphasizes that the increase in amplification with decreasing mean shear-wave velocity is distinctly less for short-period motion than for intermediate-, long- or mid-period motion. This important observation suggests that site response can best be characterized by two factors, one for the short-period component of motion and one for the other period bands. This important result is most apparent for sites underlain by soft soils. It implies that average horizontal response characteristics at the sites can be summarized by amplification factors expressed as continuous functions of mean shear-wave velocity.

The curves for the short- (0.1-0.5 s) and the mid- (0.4-2.0 s) period bands provide empirical estimates of the short-period ($F_a$) and the mid- or long-period ($F_v$) site-specific amplification factors. These amplification factors derived from the Loma Prieta strong motion data are appropriate for input ground motion levels less than or near 0.1g for sites on "Firm to hard rock", (SC-Ib). The corresponding regression curves are described by:
### Table 2b -- Short- and mid-period amplification factors* with respect to uniform ground condition SC-(II+III) (S2) for site classes defined in Table 1.

<table>
<thead>
<tr>
<th>Input Ground Motion on SC-(II+III)</th>
<th>Site Class (shear velocity, (v) (m/s))</th>
</tr>
</thead>
<tbody>
<tr>
<td>la ((g))</td>
<td>SC-la (Ao) 1620</td>
</tr>
<tr>
<td>0.1</td>
<td>0.35</td>
</tr>
<tr>
<td>0.2</td>
<td>0.25</td>
</tr>
<tr>
<td>0.3</td>
<td>0.10</td>
</tr>
<tr>
<td>0.4</td>
<td>-0.05</td>
</tr>
<tr>
<td>0.5</td>
<td></td>
</tr>
</tbody>
</table>

** Short-period Amplification Factors \(F_a\)
with respect to SC-(II+III)

<table>
<thead>
<tr>
<th>la ((g))</th>
<th>mv</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.65</td>
</tr>
<tr>
<td>0.2</td>
<td>0.60</td>
</tr>
<tr>
<td>0.3</td>
<td>0.53</td>
</tr>
<tr>
<td>0.4</td>
<td>0.45</td>
</tr>
<tr>
<td>0.5</td>
<td></td>
</tr>
</tbody>
</table>

** Mid-period Amplification Factors \(F_v\)
with respect to SC-(II+III)

* Amplification factors are predicted by \(F_a = (450 / v)^{-ma}\) and \(F_v = (450 / v)^{-mv}\) (see text, equation 2).

** \(F_a\) and \(F_v\) factors for SC-IV are scaled from corresponding factors for SC-IV (Table 2a) inferred from Loma Prieta strong motion data for la = 0.1g and numerical modelling results for la > 0.1g (R. Seed and R. Dobry, pers. commun., 1992).
<table>
<thead>
<tr>
<th>Station</th>
<th>Geologic Site</th>
<th>H.Dist.</th>
<th>S vel.</th>
<th>KJF Norm. Factors</th>
<th>Vertical period bands (secs)</th>
<th>Horizontal period bands (secs)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unit</td>
<td>Class</td>
<td>km</td>
<td>m/s</td>
<td>Vert.</td>
<td>Horizontal</td>
</tr>
<tr>
<td>South San Francisco</td>
<td>KJfsf</td>
<td>lb</td>
<td>85</td>
<td>910</td>
<td>2985</td>
<td>1.00</td>
</tr>
<tr>
<td>Yerba Buena</td>
<td>KJfsf</td>
<td>lb</td>
<td>97</td>
<td>880</td>
<td>2888</td>
<td>1.00</td>
</tr>
<tr>
<td>Rincon Hill</td>
<td>KJfsf</td>
<td>lb</td>
<td>98</td>
<td>745</td>
<td>2444</td>
<td>1.00</td>
</tr>
<tr>
<td>Pacific Heights</td>
<td>KJfsf</td>
<td>lb</td>
<td>98</td>
<td>745</td>
<td>2444</td>
<td>1.00</td>
</tr>
<tr>
<td>Diamond Heights</td>
<td>KJfsf</td>
<td>lb</td>
<td>94</td>
<td>745</td>
<td>2444</td>
<td>1.00</td>
</tr>
<tr>
<td>Piedmont Jr. High</td>
<td>KJfsf</td>
<td>lb</td>
<td>94</td>
<td>745</td>
<td>2444</td>
<td>1.00</td>
</tr>
<tr>
<td><strong>MEAN (KJF norm.: SC-lb)</strong></td>
<td></td>
<td></td>
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</tr>
<tr>
<td><strong>MEAN (&quot;ROCK&quot;) SC-lb, SC-II)</strong></td>
<td></td>
<td></td>
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</tbody>
</table>
Table 3b -- Average spectral ratios for specified period bands for "soil" sites as normalized to local rock stations and adjusted to a common ground condition ("firm to hard rock", KJF; from Borcherdt and Glassmoyer, 1992).

<table>
<thead>
<tr>
<th>Station</th>
<th>Geologic Unit</th>
<th>Site</th>
<th>H.Dist.</th>
<th>S Vel.</th>
<th>KJF Norm. Factors</th>
<th>Vertical period bands (secs)</th>
<th>Horizontal period bands (secs)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Class</td>
<td>km</td>
<td>m/s</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.1-0.5 other</td>
<td>0.1-0.5 0.5-1.5 1.5-5.0 0.1-5.0 0.4-2.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.1-0.5 0.5-1.5 1.5-5.0 0.1-5.0 0.4-2.0</td>
</tr>
<tr>
<td>Hayward BART Station</td>
<td>Qpa</td>
<td>III</td>
<td>74</td>
<td>365</td>
<td>1197</td>
<td>1.53</td>
<td>1.25 1.42 4.61 1.53 1.13 4.04 2.11</td>
</tr>
<tr>
<td>Oakland Office Bldg</td>
<td>Qpa/Qpe</td>
<td>III</td>
<td>93</td>
<td>315</td>
<td>1033</td>
<td>1.00</td>
<td>1.00 1.00 5.20 3.20 2.40 4.80 3.40</td>
</tr>
<tr>
<td>Fremont</td>
<td>Qpa</td>
<td>III</td>
<td>58</td>
<td>285</td>
<td>935</td>
<td>1.53</td>
<td>1.25 1.42 2.42 1.59 2.78 3.33 1.74</td>
</tr>
<tr>
<td>Mission San Jose</td>
<td>Qpa</td>
<td>III</td>
<td>57</td>
<td>285</td>
<td>935</td>
<td>1.53</td>
<td>1.25 1.42 3.18 1.58 2.00 2.92 1.94</td>
</tr>
<tr>
<td>Murl School (APEL 2E)</td>
<td>Qpa</td>
<td>III</td>
<td>73</td>
<td>280</td>
<td>918</td>
<td>1.53</td>
<td>1.25 1.42 4.22 1.62 1.70 3.75 1.62</td>
</tr>
<tr>
<td>MEAN (Qpa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.93 1.90 2.00 3.57 2.16</td>
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<td>STANDARD DEVIATION</td>
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<td></td>
<td></td>
<td></td>
<td>1.12 0.73 0.64 0.97 0.72</td>
</tr>
<tr>
<td>Richmond City Hall</td>
<td>Qhaf</td>
<td>III</td>
<td>109</td>
<td>288</td>
<td>945</td>
<td>1.00</td>
<td>1.00 1.00 4.00 3.03 2.51 3.80 2.94</td>
</tr>
<tr>
<td>Sunnyvale</td>
<td>Qhaf</td>
<td>III</td>
<td>46</td>
<td>268</td>
<td>879</td>
<td>1.53</td>
<td>1.25 1.42 3.98 1.68 3.14 3.63 1.88</td>
</tr>
<tr>
<td>Agnew State Hosp</td>
<td>Qhaf</td>
<td>III</td>
<td>44</td>
<td>240</td>
<td>787</td>
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<td>96</td>
<td>251</td>
<td>823</td>
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<td>1.00 1.00 4.10 3.40 5.30 4.00 4.00</td>
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<td>98</td>
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<td>626</td>
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<td>1.00 1.00 2.59 2.71 3.15 2.63 3.18</td>
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<tr>
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<td>IV</td>
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<td>1.00 1.00 0.83 0.31 0.44 0.74 0.45</td>
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<td>Qaf/Qhbm</td>
<td>IV</td>
<td>66</td>
<td>130</td>
<td>426</td>
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<td>1.25 1.42 2.68 1.96 1.19 2.52 2.30</td>
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<tr>
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<td>1.00 1.00 3.56 1.22 1.64 3.13 1.32</td>
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<td>67</td>
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<td>1.08 0.77 1.10 0.97 0.84</td>
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Table 4 -- Amplification capability for simplified site classes defined in Table1. Corresponding short-period (Fa) and mid-period (Fv) amplification factors and intensity increments are expressed with respect to site class "Firm to hard" rock, SC-Ib.

<table>
<thead>
<tr>
<th>Site class</th>
<th>Physical Description</th>
<th>Shear velocity*</th>
<th>Amplification**</th>
<th>Amplification Capability</th>
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<td></td>
<td></td>
<td>Min ft/s m/s</td>
<td>Max ft/s m/s</td>
<td>Fa Fv</td>
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<td>SC-I</td>
<td>Rocks</td>
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<td>SC-Ia (Ao)</td>
<td>Hard rocks</td>
<td>4600 1400</td>
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<td>0.9 0.8</td>
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<td>SC-Ib (A)</td>
<td>Firm to hard rocks</td>
<td>2300 700</td>
<td>4600 1400</td>
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</tr>
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<td>SC-II (B)</td>
<td>Gravelly soils and soft to firm rocks</td>
<td>1230 375</td>
<td>2300 700</td>
<td>1.3 1.5</td>
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<td>SC-III (C)</td>
<td>Stiff clays and sandy soils</td>
<td>660 200</td>
<td>1230 375</td>
<td>1.6 2.3</td>
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<tr>
<td>SC-IV</td>
<td>Soft soils</td>
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<td>SC-IVa (D)</td>
<td>Non-special study soft soils</td>
<td>660 200</td>
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<td>2.0 3.5</td>
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<tr>
<td>SC-IVb (E)</td>
<td>Special study soft soils</td>
<td></td>
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</tbody>
</table>

* Mean shear velocity \( v \) to 30 m depth.

** \( Fa = (1050/v)^*0.35 \) and \( Fv = (1050/v)^*0.65 \) as inferred from Loma Prieta strong-motion data for input ground motion levels near 0.1 g. Amplification factors agree with those derived independently by R. Dobry (1992) and R. Seed (1992).
Figure 15: Maps of San Francisco County (a) and mid San Francisco peninsula (b) showing examples of special study zones for a repeat of the 1906 earthquake on the San Andreas fault (from Borcherdt and others, 1991). The maps specify special study areas as those areas with a potential for ground motions greater than some specified critical level. The maps serve to identify zones that may be of concern during a repeat of the 1906 earthquake from the point of view of strong ground shaking due to nearness to the causative fault and or amplification by geologic deposits.
Figure 14 Maps of San Francisco County (a) and mid San Francisco peninsula (b) showing zones delineated according to their potential for ground shaking to exceed specified levels expressed in terms of earthquake intensity for a repeat of the 1906 earthquake on the San Andreas fault (from Borcherdt, and others 1991). The maps illustrate the concept of exceedent potential maps. They represent the superposition of maps illustrating exceedent opportunity (Figure 13) and amplification capability (Figure 12). Contour levels are expressed in terms of earthquake intensity with numbers 4 - 0 corresponding to letters A - E of the 1906 San Francisco intensity scale.
Figure 13 Maps of San Francisco County (a) and mid San Francisco peninsula (b) showing zones delineated according to specified levels that ground shaking (earthquake intensity) may exceed on a uniform ground condition (firm to hard rock) for a repeat of the 1906 earthquake on the San Andreas fault (from Borcherdt, and others, 1991). The maps illustrate the concept of exceedent opportunity maps, if the contours are specified in terms of an appropriate return period for the event. They are based on an empirical attenuation relation derived from reliable 1906 intensity data (Borcherdt, and others, 1975). Numbers 4 - 0 correspond to letters A - E on the 1906 San Francisco intensity scale.
Figure 12 Maps of San Francisco County (a) and mid San Francisco peninsula (b) showing zones delineated according to their capability to amplify ground shaking (from Borcherdt and others, 1991). The maps are based on the classification of mapped geologic material units into classes of which the four major classes correspond to the simplified site classes defined in Table 1. Subclassifications correspond to geologic differentiations with only slight distinctions in amplification capabilities (see Wentworth, and others, 1991) and Borcherdt, and others 1991).
Figure 10. (a) Short-period $F_s$ and (b) mid-period amplification factors with respect to combined site class SC-(II+III) plotted with logarithmic scales as a continuous function of mean shear-wave velocity using the indicated equations for specified levels of input ground motion. Amplification factors with respect to SC-(II+III) for the simplified site classes are shown for the corresponding mean shear-wave velocity interval for input ground motion levels near 0.1g.
Figure 9. (a) Short-period $F_s$ and (b) mid-period $F_m$ amplification factors with respect to "firm to hard rock" (SC-Ib) plotted with logarithmic scales as a continuous function of mean shear-wave velocity using the indicated equations for specified levels of input ground motion. Amplification factors with respect to SC-Ib for the simplified site classes are shown for the corresponding mean shear-wave velocity interval for input ground motion levels near 0.1g.
Figure 8. Short-period $F_a$ and mid-period $F_v$ amplification factors with respect to firm to hard rock (SC-Ib) plotted as a continuous function of mean shear-wave velocity using the regression equations derived from the Loma Prieta earthquake. The 95% confidence intervals for the ordinate to the true population regression line and the limits for ±2 standard error of estimate are shown. Corresponding amplification factors predicted with respect to firm to hard rock for the simplified site classes also are shown.
Figure 7. Average horizontal amplification factors with respect to “firm to hard rock” (SC-Ib) plotted as linear functions of mean shear velocity for the short-period, mid-period, intermediate-period, and long-period bands as inferred from the Loma Prieta strong-motion data (see Figure 6 for plots with logarithmic scales). The plots emphasize the need for both a short-period and mid- or long period amplification factor to account for the response characteristics of near surface deposits.
Figure 6. Average horizontal amplification factors, regression curves, and confidence intervals inferred from the free-field strong-motion recordings of the Loma Prieta earthquake of October 17, 1989 for the (a) short- (0.1-0.5 s), (b) intermediate- (0.5-1.5 s), (c) long- (1.5-5.0 s) and (d) mid- (0.4-2.0 s) period bands. The 95% confidence intervals for the ordinate to the true population regression line and the limits for ± 2 standard error of estimate are shown.
Figure 5. (a) Short-period $F_a$ and (b) mid-period $F_v$ amplification factors with respect to combined site class SC-(II+III) as a continuous function of mean shear-wave velocity using the indicated equations for specified levels of input ground motion. Amplification factors with respect to SC-(II+III) for the simplified site classes are shown for the corresponding mean shear-wave velocity interval for input ground motion levels near 0.1g.
Figure 4. (a) Short-period $F_s$ and (b) mid-period $F_v$ amplification factors with respect to firm to hard rock (SC-Ib) plotted as a continuous function of mean shear-wave velocity using the indicated equations for specified levels of input ground motion. Amplification factors with respect to SC-Ib for the simplified site classes are shown for the corresponding mean shear-wave velocity interval for input ground motion levels near 0.1g.
Figure 3b. Shear-wave velocities for depth intervals determined in bedrock materials in the Los Angeles region (from Fumal and Tinsley, p. 140, 1985). The materials are divided into groups according to fracture spacing, hardness, and lithology (see figure caption 2b for definitions). Mean and standard deviation (bar) for interval velocities in each group are shown. Intervals for mean shear-wave velocity to 30 m (100 ft) for simplified site classes are designated.
Figure 3a. Shear-wave velocities for depth intervals determined in unconsolidated to moderately consolidated deposits (soils) in the Los Angeles region (from Fumal and Tinsley, p. 132, 1985). The deposits are divided into groups according to texture and standard penetration resistance N (blows per foot). Mean and standard deviation (bar) for interval velocities in each group are shown. Intervals for mean shear-wave velocity to 30 m (100 ft) for simplified site classes are designated.
Figure 2b. Shear-wave velocities for depth intervals determined in bedrock materials in the San Francisco Bay region (from Fumal 1978; Borcherdt et al., 1978). The materials are divided into groups according to fracture spacing, hardness, and lithology. Mean and standard deviation (bar) for interval velocities in each group are shown. Intervals for mean shear-wave velocity to 30 m (100 ft) for simplified site classes are designated. (Fracture spacing is defined as very close 0 to 1 cm, close 1 to 5 cm, moderate 5 to 30 cm, and wide 30 to 100 cm. Hardness is defined as response to geologic hammer; hard, hammer bounces with solid sound; firm, hammer dents with thud; soft, hammer pick penetrates.)
Figure 2a. Shear-wave velocities for depth intervals determined in unconsolidated to moderately consolidated deposits (soils) in the San Francisco Bay region (from Fumal 1978; Borcherdt et al., 1978). The deposits are divided into groups according to texture and standard penetration resistance N (blows per foot). Mean and standard deviation for interval velocities in each group are shown. Intervals for mean shear-wave velocity to 30 m (100 ft) for simplified site classes are designated.
Figure 1. Site-specific response spectra defined in terms of short-period and mid-period input ground-motion levels $I_a$ and $I_v$ and amplification factors $F_a$ and $F_v$. 

$S_A = \text{Minimum} \left\{ \frac{I_a \times F_a}{I_v \times F_v \times (1/T)} \right\}$
NEW DEVELOPMENTS IN ESTIMATING BASIN RESPONSE EFFECTS ON GROUND MOTIONS

Arthur Frankel
U.S. Geological Survey, MS 966, Box 25046, DFC, Denver, CO 80225

Abstract

Damage to structures located in sedimentary basins can be caused by horizontally-propagating seismic waves trapped in the alluvium, in addition to near-vertically propagating shear waves. I describe recent observations from arrays of seismometers which document the presence of short-period (0.5-5 sec) surface waves trapped in alluvial basins, even for relatively close-in earthquakes. These surface waves are produced by conversion of S-waves at the edges of the basins. The surface waves have slow horizontal velocities of propagation (≤ 1 km/sec). A nearby large earthquake is likely to generate long-duration (tens of seconds) surface waves with sizable amplitudes that are potentially hazardous to structures with natural periods of about 0.5 sec and longer, such as tall buildings, elevated pipelines, bridges, and base-isolated buildings. I emphasize the inadequacy of using flat-layered models for synthesizing ground motions in sedimentary basins, which do not produce these large surface waves and, thus, underestimate the duration and amplitude of shaking at these periods. We are using computer simulations of seismic wave propagation in 3-D models of sedimentary basins to identify areas of surface wave focusing in actual basins and to produce site-specific time histories of ground motions for future large earthquakes.

Basin-Surface Waves

The mitigation of earthquake hazard requires understanding seismic wave propagation in sedimentary basins. Here, "sedimentary basins" refers to valleys filled with thick sequences of unconsolidated materials such as sands, clays, and gravel. Many major population centers are located in sedimentary basins near the rupture zones of expected large earthquakes or near sources of seismic hazard. Examples in the United States include: Los Angeles, San Bernardino, San Jose, San Francisco, Oakland, Seattle, Portland, Memphis, St. Louis, and Boston.

Figure 1 shows a schematic vertical cross-section of a sedimentary basin adjacent to a hypocenter of an earthquake. S-waves from the hypocenter impinge on the alluvium-bedrock interface beneath the site. Because of the low-velocity of the alluvium relative to the bedrock, the ray path of the S-waves will be bent so that they propagate nearly vertically in the alluvium under the site. These S-waves will be amplified by the low rigidity of the alluvium. In addition, there will be multiple reflection of the vertically-propagating S-wave between the free surface and the bedrock-alluvium.
interface, causing resonance at certain periods related to the thickness and shear-wave velocity of the alluvium.

There is more to seismic wave propagation in sedimentary basins than the amplification and resonance of vertically-propagating S-waves. When S-waves from the hypocenter are incident on the edge of the basin, they can convert to surface waves that travel horizontally across the basin. These surface waves have relatively slow velocities of horizontal propagation ($\leq 1$ km/sec) and arrive after the direct S-wave for sites in the basin. These surface waves typically have periods of 0.5 to 5 sec or longer. They can constitute the largest arrivals at these periods in time histories of ground motions at alluvial sites. Models with a horizontal interface between the alluvium and the bedrock cannot produce the S-wave to surface wave conversion.

Perhaps the most significant aspect of these surface waves is their long duration, typically measured in tens of seconds. Since these waves are trapped in the alluvium, their motions are limited largely by the damping (internal friction) of the deep sediments. The surface waves can travel to the edges of the basin and much of their energy can be reflected back into the basin for another transit. In some cases where the width of the basin is comparable to its depth, multiple-reflections can set up resonances of the whole basin, which is the bowl of jello analogy often mentioned to describe sedimentary basin response during large earthquakes.

Obviously, the presence of large, basin surface waves with durations of tens of seconds can have a strong effect on the vulnerability of tall buildings (5 stories or more), bridges, and other structures with natural periods of 0.5 sec or longer. There is a growing effort for base isolation of important structures. These base isolators typically have natural periods of 2-3 sec, similar to the dominant periods of observed basin surface waves. Therefore, I think the long duration of these surface waves should be considered in the design of base isolation systems.

There have been observations of basin surface waves during several large earthquakes. Some studies have attributed the damage to Mexico City from the 1985 earthquake to short-period surface waves trapped in the lake-bed sediments (e.g., Kawase and Aki, 1989). Large, short-period surface waves were observed in the San Fernando Valley and Los Angeles basin from the 1971 San Fernando Earthquake (see Vidale and Helmberger, 1988). The damage to the city of Leninakan during the 1988 Armenian earthquake may have been partially caused by basin surface waves, since large surface waves were observed in aftershock recordings from a site in that city (Frankel et al., 1989). Large surface waves with periods of 1-5 sec were observed in the Santa Clara Valley from the Loma Prieta earthquake and its aftershocks (Frankel et al., 1991).

Figure 2 (top) shows an example of basin surface waves observed for a M4.9 aftershock of the Big Bear earthquake that occurred in 1992 in southern California. This recording is from a deep-soil site in the San Bernardino Valley, about 44 km from the
The figure shows the transverse horizontal component of acceleration after frequencies higher than 1 Hz have been filtered out of the record. The long duration of motion is striking: arrivals 60 sec after the direct S-wave have similar amplitudes as the initial S-wave arrival. This long-duration motion was not observed at an adjacent rock site. The duration of the source rupture process is only about 1 sec. This seismogram is from a station in a three-station array with a station spacing of about 200m. This array was used to determine the apparent velocity and back azimuth of arrivals in the seismogram (Figure 2). Apparent velocity is the velocity of propagation of a specific phase across the free surface. Back azimuth is the direction the waves are propagating from. The phases in the first 30 sec of the seismogram are arriving from the direction of the source. In contrast, the phases from 50 to 80 sec in the seismogram are arriving from directions quite different from the source, some from the opposite direction (Figure 2). The apparent velocities of these late arrivals range from about 1-3 km/sec. Therefore, much of this later-arriving energy are basin surface waves with slow apparent velocities traveling in various directions across the San Bernardino Valley. Some of these surface waves were probably reflected from the edges of the valley. This aftershock had a shallow depth of about 1 km. Other deeper events also generate these basin surface waves.

Theoretical Modeling

There has been substantial progress in theoretical modeling of basin surface waves over the past 15 years. Most of the theoretical modeling consists of computer simulations of seismic wave propagation in 2-D and 3-D models of sedimentary basins. 2-D models represent a vertical cross section of the basin. Until recently, 3-D simulations required too much computer time and memory to use grids large enough to propagate waves at frequencies of engineering interest. Large-scale 3-D simulations are now feasible on fast workstations, as well as supercomputers and parallel processors. The simulations usually employ the finite difference or finite element techniques. Typical simulations use realistically-complex models of the velocity and density structure of basins. Probably the biggest limit to our understanding wave propagation in basins is the lack of detailed information on the structure and properties of these basins.

The accompanying paper describes 3-D simulations of ground motions for the San Bernardino Valley in southern California for hypothetical earthquakes on the San Andreas faults. The USGS has also done 3-D simulations for the San Jose area, for an aftershock of the Loma Prieta earthquake (Frankel and Vidale, 1992). The goal of our work is to predict site-specific time histories of ground motions in sedimentary basins for future large earthquakes. We also want to identify, in advance, areas in these valleys where surface wave energy will be focused in future earthquakes.
References


Figure 1. Schematic vertical cross section of sedimentary basin, showing hypocenter (filled circle) of earthquake and raypaths. Idealized seismograms for two sites (inverted triangles) are shown.
Figure 2. Results of array analysis of transverse records of M4.9 aftershock of 1992 Big Bear earthquake, recorded in San Bernardino Valley. (Top) One of the three transverse acceleration records (low-pass filtered at 1 Hz) used in the analysis. "S" denotes direct S-wave arrival. (Middle) Apparent velocity of arrivals determined from moving window analysis. (Bottom) Back azimuth of arrivals determined from moving window analysis. Back azimuth is in degrees clockwise from North. The horizontal line in the bottom panel is the back azimuth to the epicenter.
Three-dimensional finite difference simulations of elastic waves in the San Bernar-
dino Valley were performed for two hypothetical earthquakes on the San Andreas
fault: a point source with moment magnitude M5 and an extended rupture with M6.5. A
method is presented for incorporating a source with arbitrary focal mechanism in the
grid. Synthetics from the 3-D simulations are compared with those derived from 2-D
(vertical cross section) and 1-D (flat-layered) models. The synthetic seismograms from
the 3-D and 2-D simulations exhibit large surface waves produced by conversion of
incident S-waves at the edge of the basin. Seismograms from the flat-layered model
do not contain these converted surface waves and underestimate the duration of shak-
ing. The seismograms from the 3-D simulations have larger amplitude coda than the
seismograms from the 2-D case because of the presence of off-azimuth surface wave
arrivals in the 3-D simulations that are not included in the 2-D simulations. Snapshots
of the wavefield of the 3-D simulation show that these off-azimuth arrivals represent
surface waves reflected from the edges of the basin. The anelastic attenuation of the
sediments is a key parameter controlling the overall duration of motion. Some of the
coda energy at rock sites near the basin edges represents leakage of surface wave
energy out of the basin. For the M6.5 earthquake simulation, the largest ground velo-
cities occur where surface waves reflected from the edge of the basin interfere con-
structively with the trapped waves that follow the direct S-wave. Maps of maximum
ground velocity are produced for two directions of rupture propagation. The largest
velocities occur in localized portions of the basin. The location of the largest veloci-
ties changes with the rupture propagation direction. Contours of maximum shaking are
also dependent on asperity positions and radiation pattern.

Introduction

The mitigation of earthquake hazard requires understanding seismic wave propa-
gation in sedimentary basins. Many major population centers are located in sedi-
mentary basins near the rupture zones of expected large earthquakes. Several theoretical
studies over the past ten years or so using 2-D numerical methods have demonstrated the importance of body wave to surface wave conversions at the edges of the basins (e.g., Bard and Bouchon, 1980; Harmsen and Harding, 1981). These surface waves prolong the duration of motion at long periods ($\geq$ 1sec) over that expected from 1-D models using flat, horizontal layers. There are observations of sustained, long-period motions in sedimentary basins that appear to be surface wave energy produced by conversion of body waves near the basin edges (e.g., Vidale and Helmberger, 1988; Frankel et al., 1991). The large amplitude and long duration of these surface waves pose a significant and under-appreciated hazard to man-made structures with natural periods of 1 sec or larger, such as buildings of 10 or more stories, some highway overpasses, and elevated pipelines.

There is growing observational evidence that later arrivals in seismograms recorded in sedimentary basins are surface waves scattered from the edges of the basin, highlighting the need for 3-D simulations. Frankel et al. (1991) used a dense array to show that later arrivals in seismograms of Loma Prieta aftershocks consisted of surface waves traveling various directions across the Santa Clara Valley. Phillips et al. (1992) reported observations of Love waves in the Kanto basin, Japan, produced by scattering of S-waves at one location near the edge of the basin. Spudich and Iida (1992) found that coda waves of aftershocks of the North Palm Springs earthquake were scattered from specific locations in the Coachella Valley.

Previous work on 3-D simulations in realistic basin models has been limited. Graves and Clayton (1992) simulated acoustic wave propagation in 3-D ellipsoidal basin structures in southern California using a paraxial approximation. They found strong patterns of focusing and defocusing. Toshinawa and Ohmachi (1992) used a 3-D finite element method to model Love wave propagation in the Kanto plain, Japan, for waves with periods of about 10 sec. They found that the 3-D simulation produced synthetics with larger amplitude and duration than 2-D simulations and that the 3-D synthetics agreed better with the data. Frankel and Vidale (1992) used 3-D finite difference simulations to study elastic wave propagation in the Santa Clara valley, California, for frequencies up to 1 Hz. This study showed how body wave to surface wave conversion occurred over an irregular basin margin and how surface waves could be scattered back into the basin from the edges of the valley. Yomogida and Etgen (1993) performed a 3-D finite difference simulation of elastic waves in the Los Angeles basin for the Whittier-Narrows earthquake. Their simulation reproduced some of the observed amplification pattern in the basin.

One purpose of this paper is to illustrate the propagation effects that occur in 3-D basin models with realistically-complex geometries. I show how synthetics derived from 3-D simulations are significantly different from those obtained from 2-D and 1-D velocity models. The second purpose of this paper is to describe how extended earthquake sources can be incorporated in the finite difference method. I demonstrate how
rupture directivity affects the pattern of maximum ground motions in the basin.

I chose to simulate ground motions in the San Bernardino Valley partly because it is a major population and industrial center located near an expected large earthquake on the San Andreas fault (Working Group on California Earthquake Probabilities, 1988). The relatively small size of the San Bernardino basin makes it feasible to use a 3-D simulation at frequencies of engineering significance (up to 1 Hz). This study is a first step towards predicting the amplitudes, spectra and durations of ground motions in the San Bernardino Valley for a future large earthquake on the adjacent segment of the San Andreas fault.

Basin Model

A crude model of depth to bedrock was constructed for the San Bernardino valley from water well, oil well, and limited refraction data. Figure 1 shows the area of the simulation for the San Bernardino Valley. The San Andreas fault constitutes the upper border of the alluvial basin. In the basin, unconsolidated Quaternary and Tertiary continental deposits overlie consolidated Tertiary deposits, which overlie a pre-Tertiary basement complex of igneous and metamorphic rocks (Dutcher and Garrett, 1963). The crosses in Figure 1 represent measurements of depth to bedrock. The largest values (1008, 960 and 914 m) were depth to basement obtained from a refraction line reported by Hadley and Combs (1974). Most of the other measurements were depth of bedrock from water well data compiled by Dutcher and Garrett (1963). Water wells in near the center of the valley do not reach bedrock. A few measurements were from oil well data mentioned in Youngs et al. (1982). The 457m value shown in Figure 1 represents the maximum depth of a water well that did not reach bedrock.

The model contains a vertical offset in alluvial thickness at the San Jacinto Fault (D. Morton, pers. comm., 1991; Dutcher and Garrett, 1963). The amount of this vertical offset is not known, although it is thought to increase to the south. The San Jacinto fault in this area represents another likely source of damaging earthquakes in the future.

Developing a model of seismic velocities for the basin was hindered by the sparse depth to bedrock data. I added some conjectured depth to bedrock values near the edges of the valley. The observed and conjectured depth to bedrock values were fit by a minimum-tension surface, resulting in the contours shown in Figure 1. These were the depth to bedrock values used in the simulation.

There are several outcrops of bedrock within the San Bernardino valley. The detailed outlines of these outcrops were digitized from a geologic map (California Division of Mines and Geology, 1969) and included in the model. The largest outcrop forms the Shandin Hills (Figure 1).
I characterized the alluvium by one set of velocities and density. Although the finite difference method can accommodate layering within the alluvium, I felt our limited knowledge of the velocity structure did not warrant a more complex model. I assigned a shear wave velocity of 0.6 km/sec, a P-wave velocity of 1.1 km/sec, and density of 2.0 g/cc for the alluvium. The shear wave velocity is typical to that found for Quaternary alluvium below 100m or so (Joyner et al., 1991). The P-wave velocity is lower than that reported for alluvium (2.0 km/sec) because using a higher $V_P/V_S$ ratio produces a numerical instability due to the free surface (see Vidale and Clayton, 1986). Using the lower than actual P-wave velocity decreases somewhat the group velocity of the Airy phase of the Rayleigh wave (see Frankel and Vidale, 1992). For the bedrock below the alluvium I used a shear wave velocity of 2.0 km/sec, P-wave velocity of 3.5 km/sec and density of 2.6 g/cc. These shear wave velocities were chosen as a compromise between those expected for consolidated sedimentary rocks and metamorphic basement rocks. A horizontal interface was placed at 3 km depth where the shear wave and P-wave velocities increased to 2.9 km/sec and 5.0 km/sec, respectively, while the density remained at 2.6 g/cc. This P-wave velocity is somewhat lower than that found for basement rocks in the region by Hadley and Combs (1974; 5.3 km/sec).

3-D Finite Difference Method With Arbitrary Earthquake Source Mechanism

The finite difference method is used to propagate the complete elastic wavefield through a 3-D grid with heterogeneous P and S-wave velocities and density. Details of the 3-D finite difference scheme are described in Frankel and Vidale (1992). The method used here is based on the elastic wave equations for displacement in a heterogeneous medium. The scheme is accurate to fourth order in space and second order in time. Anelastic attenuation is not included in the simulation but can be applied to the synthetics (see below). Absorbing boundary conditions (Clayton and Engquist, 1977) were applied on all sides of the grid, except the free surface.

The grid spacing was 100m for both the horizontal and vertical directions. The simulation corresponds to a volume 37 km by 16 km by 7 km in depth. This involves about 4.1 million grid points. The time step was 0.012 sec. Most of the simulations described here were performed on a Cray YMP supercomputer, although a 23 MFLOPS (million of floating point operations per second) workstation was used for one of the simulations. Previous studies have found that the fourth order finite difference code requires about 6 grid points per wavelength for adequate accuracy (e.g., Alford et al., 1974). Since the slowest shear wave velocity is 0.6 km/sec, the synthetics are accurate up to 1 Hz (100m grid spacing). Frankel and Vidale (1992) compared synthetic seismograms from the 3-D finite difference and reflectivity methods for a horizontal layer over a half space. They reported that the synthetics from the 3-D finite difference method have about 10% longer durations than those.
derived from a reflectivity method, because numerical inaccuracies in the finite difference method tend to make the phase velocity slightly slower than the proper value. Thus, the synthetics shown in this paper probably overestimate the duration by about 10%. However, since the S-wave velocities of the shallow alluvium used in the simulation are greater than the actual velocities, the duration of the synthetics may underestimate those of observed seismograms. Frankel and Vidale (1992) also noted that the peak amplitude of the seismogram produced by the 3-D finite difference method was about 80% of that from the reflectivity method, for the case of a horizontal layer over a half space.

I considered two hypothetical earthquakes along the San Andreas fault: a moment magnitude M5 (point source) and a M6.5 earthquake which ruptures a 30 km long segment of the fault (Figure 2). The point source was simulated by specifying displacements at the six grid points surrounding the source location. These displacements correspond to the equivalent force system for an earthquake with the desired focal mechanism. The earthquakes had right-lateral strike-slip motion on a vertical fault. The extended rupture was simulated by firing off point sources with a delay appropriate for propagating rupture (0.8 times the S-wave velocity). For both earthquakes, the shape of the slip velocity function at any point on the fault was a Gaussian with a width at 1/e amplitude of 1 sec. The tails of the Gaussian were truncated at ±1.5 sec from the peak. This slip velocity function produced a far-field velocity pulse with a dominant period of about 2.5 sec, for a point source. This dominant period corresponds to a wavelength of 15 grid spacings for S-waves in the alluvium. This long wavelength effectively suppresses numerical artifacts such as diffractions from stairsteps of sloping interfaces.

It is necessary to determine how the displacements applied at grid points around a point source are related to the seismic moment. Consider one of the three, coupled elastic wave equations for the case of an internal force:

$$\rho \frac{\partial^2 u}{\partial t^2} = g(u,v,w,\lambda,\mu) + f_x$$

Here, \(u,v,w\) are the displacements in the \(x,y,z\) directions, respectively. \(\rho\) is density. \(g(u,v,w,\lambda,\mu)\) is a function of the spatial derivatives of \(u,v,w\), and the Lame constants \(\lambda\) and \(\mu\). \(f_x\) is the force/volume in the \(x\)-direction for this time step, representing one of the equivalent forces of the earthquake. We need to relate this term to the moment of the earthquake. The partial derivative on the left hand side can be approximated by the finite difference expression

$$\frac{\partial^2 u}{\partial t^2} \approx \frac{u(1)-2u(0)+u(-1))}{(\Delta t)^2}$$

where \(u(1)\) is the value of \(u\) at the next time step, \(u(0)\) is the value at the present time step and \(u(-1)\) is the value at the previous time step. \(\Delta t\) is the time step. Substituting equation (2) into equation (1) and solving for \(u(1)\) yields...
\[ u(1) = \frac{(\Delta t)^2}{\rho} \left[ g(u,v,w,\lambda,\mu) + f_x \right] + 2u(0) - u(-1) \]  \hspace{1cm} (3)

Consider the displacement component \( u \) to be added at the two grid points located ±1 grid point in the \( x \) direction from the source location. This displacement is proportional to the \( M_{xx}(t) \) component of the moment tensor, which is proportional to the fault slip at time \( t \). For these grid points, the equivalent force of the earthquake in the \( x \)-direction equals \( M_{xx}(t)/2h \), where \( h \) denotes the grid spacing. Since the force is applied at one grid point, the appropriate volume is one grid cell or \( h^3 \). The force per unit volume applied at that grid point is then

\[ f_x = \frac{M_{xx}(t)}{2h^4} \]

Substituting this into equation (3) results in

\[ u(1) = \frac{(\Delta t)^2}{\rho} \left[ g(u,v,w,\lambda,\mu) + 2u(0) - u(-1) \right] + \frac{(\Delta t)^2 M_{xx}(t)}{2h^4 \rho} \]

The rightmost term is the displacement in the \( x \) direction that must be applied at that grid point to simulate an earthquake with moment tensor component \( M_{xx}(t) \). In general, at time \( t \) the vector displacement \( U(t) \) added to each of the 6 grid points adjacent to the source location is

\[ U(t) = \frac{(\Delta t)^2 A(t)}{2h^4 \rho} \]

\( A(t) \) is a vector dependent on the grid point considered and the appropriate components of the moment tensor. For the grid point one grid spacing in the +x-direction from the source

\[ A(t) = M_{xx}(t)x + M_{xy}(t)y + M_{xz}(t)z \]

For the grid point one grid spacing in the +y-direction from the source

\[ A(t) = M_{xy}(t)x + M_{yy}(t)y + M_{yz}(t)z \]

For the grid point one grid spacing in the +z-direction from the source

\[ A(t) = M_{xz}(t)x + M_{zy}(t)y + M_{zz}(t)z \]

\( x,y,z \) denote the unit vectors in the \( x,y,z \) directions. The convention follows Aki and Richards (1980). For a grid oriented north-south and east-west, positive \( x \) is northward, positive \( y \) is eastward, and positive \( z \) is downward. At the 3 grid points in the negative \( x,y,z \) directions from the source, \( A(t) \) is the negative of the respective values given above.
Figure 3 shows synthetic seismograms (velocity) at site 1 for the M5 simulation. The source is located at 6 km depth. The first phase in the transverse synthetics is the direct S-wave. The largest phase in the transverse synthetic is a phase that has been reflected twice from the free surface and the alluvium-bedrock interface. A synthetic ground motion seismogram was calculated using the reflectivity method for horizontal layers (1-D model). The 1-D model consisted of three layers with the same velocities and densities used in the 3-D model. I used an alluvial thickness of 950m for site 1 (Figure 2). In Figure 3, the largest phase in the 1-D synthetic is the direct S-wave. Reflections from phases multiply reflected at the free surface and alluvium-bedrock interface are smaller than the direct wave. Note also that a 1-D model would not predict any motion on the radial and vertical components for site 1, which is located at an azimuth 90° from the strike of this strike-slip earthquake. The 3-D simulation shows significant motions on the radial and vertical components.

A 2-D finite difference simulation (SH) was run using a vertical cross section of the model in Figure 1 along the line connecting the receiver and the source: An equivalent point source was incorporated into the 2-D grid using the method of Helmberger and Vidale (1988). The 2-D transverse synthetic is quite similar to the 3-D synthetic for this receiver (Figure 3). The 2-D synthetic also contains the large phase that is multiply-reflected in the alluvium.

The efficient trapping of the S-wave within the alluvium is caused by the dip of the alluvium-bedrock interface away from the source. The ray paths for an incoming S-wave and its reflections within the alluvium are shown in Figure 4. As the S-wave is multiply-reflected by the free surface it has an increasingly grazing angle of incidence at the alluvium-bedrock interface. Eventually the S-wave is critically reflected within the alluvium, becoming a surface wave. At increasing distance from the source, the direct S-wave becomes more grazing at the underside of the alluvium-bedrock interface. This causes more downward reflection of the direct S at the interface, reducing the amplitude of the direct S at the surface. The dipping geometry in Figure 4 causes the multiply-reflected wave to have higher amplitude than the direct wave.

Figure 5 compares synthetic seismograms for the San Bernardino site for the 3-D, 1-D and 2-D models. This site is located near the downtown area of the city of San Bernardino. The difference between synthetics from the 3-D simulation and those from a flat-layered (1-D) model is dramatic. The 3-D simulation has a much longer duration of shaking than the 1-D model. The basic reason for this is the trapping of body waves in the alluvium that occurs in the 3-D simulation. The incident S-wave is essentially converted into surface waves (Rayleigh and Love) at the margins of the basin. These surface waves have relatively slow group velocities, and prolong the duration of shaking in the basin. Models using flat layers cannot account for these body wave to
Simulations for M6.5 Earthquakes

The 3-D synthetics have substantially more coda energy after the initial S-wave than those from the 2-D simulations (Figure 5). This is due to the presence of off-azimuth surface waves in the 3-D simulation that are not included in the 2-D modeling. By "off-azimuth," I refer to waves arriving from azimuths different from the source. The 2-D radial and vertical synthetics do show the Rayleigh wave energy produced by the S-wave conversion at the edge of the basin (at 10-14 sec and 22-48 sec; Figure 5 bottom). The phase at 20 sec in the 2-D transverse synthetic represents energy reflected from the San Jacinto fault.

The anelastic attenuation of the sediments will be a major limiting factor on the duration of shaking. I convolved the synthetics with a time varying Q-operator to approximate the effect of anelasticity in the alluvium. The amount of attenuation in the Q operator is increased with increasing time in the synthetics (see Vidale and Helmberger; Frankel and Vidale, 1992). Figure 6 shows the San Bernardino synthetics from the 3-D simulation for the cases with infinite Q and shear wave Q values of 50 and 25 in the alluvium. As the Q is decreased, later arrivals are diminished in amplitude and become longer period. The presence of large, late arrivals in observed seismograms from sedimentary basins indicates that the shear wave Q of deep sediments is probably at least 50 (see Frankel and Vidale, 1992).

Figure 7 compares the synthetics for adjacent valley and rock sites (sites 2 and 3). The duration of shaking is much longer on the valley receiver than the rock site, even though the valley site synthetic has been convolved with a set of Q operators corresponding to a shear-wave Q of 50 in the sediments. For the transverse component, the peak amplitude is about twice as large on the sediment site than the rock site, as expected from the impedance contrast between the alluvium and rock. The 3-D radial and vertical synthetics for the valley site show substantial amplitude. For horizontal layering, there should be no motion on the radial and transverse components, given the focal mechanism of the source and the azimuth of these receivers. The radial and vertical motions at the valley site are considerably larger than those on the rock site. This is due to the off-azimuth body and surface wave arrivals at the valley site. The coda after the initial S-wave arrival on the radial and transverse components at the rock site is caused by leakage of surface wave energy from the basin.

Simulations for M6.5 Earthquakes

Next, I simulated ground motions for a M 6.5 earthquake that ruptures a 30 km long portion of the San Andreas fault (see Figure 2). This earthquake occurs on a vertical fault plane over depths of 3 to 6 km, nucleating at the lower left hand corner of the fault plane. The rupture propagates at 0.8 times the shear wave velocity of the
source region. The rupture takes about 13 sec to propagate from one end of the fault to the other. To simulate asperities, I constructed a slip function whose peak amplitude varied stochastically across the fault plane (Figure 8). The slip function had a Gaussian probability distribution and a Gaussian correlation function with a correlation distance of 1 km. I chose this correlation distance so that there would be areas of localized slip (sub-events) on the fault with dimensions similar to those observed for earthquakes of this moment (see, e.g., Frankel and Wennerberg, 1989). The slip realization used here has much of its slip located near the two ends of the fault. The maximum slip on the fault probably overestimates that to be expected for an earthquake of this moment, since the slip is confined to a 3 km range of depth. This depth limitation is imposed by the grid size. Thus, the velocities found here in the simulations probably overestimate the velocities that would be generated by an earthquake of this size.

Snapshots of the wavefield are depicted in Figures 9a-d. They show the horizontal component of velocity parallel to the strike of the fault. For each figure, the top panel is the slip velocity on the fault plane at that time step. The rupture propagates from left to right. The middle panel is a map view of the ground velocities. The bottom panel is a vertical cross section of the ground velocities along the red line drawn from left to right across the model, through the location of San Bernardino.

At 12 sec into the simulation (Figure 9a), the rupture has propagated to the middle of the fault (see top panel). The wavefronts dominant at this time were radiated from the sub-event from the sub-event at the left (northwest) end of the fault. The direct S-wave is seen as the diffuse yellow areas in the center and lower left portions of the basin (labelled "S1"). The direct S-wave forms the more distinct yellow zone on the rock just northeast of the San Andreas fault. This lags behind the rupture front in map view because of the delay in propagating to the surface and the polarity reversal across the fault. The largest amplitude at this time is shown by the red band just south of the Shandin Hills. This prominent band represents the phase reflected twice at the free surface and the alluvium-rock interface. I label this phase as "A". The curvature of this wavefront corresponds to the curvature of the contour lines of depth to bedrock.

The snapshot at 9.6 sec into the simulation (Figure 9b) shows the wavefronts propagating further into the valley, south of the Shandin Hills. These wavefronts are multiple reflections between the free surface and the alluvium-bedrock interface. Also visible is a reflection of the direct S-wave by the southern edge of the valley. This is the dark blue, semi-circular phase in the lower left-center portion of the map. This trapped phase is travelling northeastward in the valley. The vertical cross section shows that the largest motions occur within the alluvium.

At 12.0 sec (Figure 9c), the large asperity on the southeast (right) end of the fault is activated. The prominent yellow-orange wavefront near the center of the valley is the phase ("A") reflected twice at the free surface and the alluvium-bedrock interface. This energy was radiated from the left end of the fault. The cross section illustrates...
that this energy is trapped in the alluvium. The dark blue wavefront behind A has been reflected three times at the alluvium-bedrock interface and twice at the free surface and is labelled "B". The reflections from the southern edge of the basin form the yellow and dark blue wavefronts in the lower left portion of the map view. Some of the dark blue area in the right side of the basin represents the reflection of the direct S-wave by the right side of the basin. These reflections are trapped waves propagating leftward back into the valley. The yellow wavefront on the left side of the map view (west of the San Jacinto fault) is an Airy phase of the Love wave produced by conversion of the incident S-wave. This Airy phase is developed by the flat basin in the model to the west of the San Jacinto fault. I identify this phase as an Airy phase because its group velocity corresponds to that expected for the Airy phase of the Love wave.

By 16.8 sec into the simulation (Figure 9d) the motion on the fault plane has been over for about 4 sec. However, large ground velocities persist in the valley. The direct S-wave for the southeast sub-event can be seen on the left-hand side of the grid, in both the map view and the cross section (labelled "S2"). The largest velocities occur in the right portion of the valley, marked by the red color. These large motions are caused by the constructive interference of three sets of arrivals. The first set are the multiply-reflected waves from the sub-event on the northeast end of the fault (phases A and B). The second set are the S to surface wave reflections from the southeast edge of the valley that are propagating leftward into the valley (labelled "C"), opposite in direction to the first set of trapped waves (A and B). The constructive interference between the rightward and leftward propagating trapped waves is one reason for the large amplitudes in this part of the valley. The third set of arrivals consists of the direct S-wave and trapped waves from the sub-event on the southeast end of the fault. Thus, the location of the maximum ground motion is influenced by the position and timing of asperities on the fault, in addition to the basin structure.

To better understand and document the different wave types in the simulation, synthetic seismograms were analyzed from a small array placed around site 1. This array consisted of nine receivers arranged in a square, 3 x 3 array. The spacing between each receiver was 200 m. The vector slowness of successive windows in the seismograms was determined using the method described in Frankel et al. (1991) and Frankel and Vidale (1992). I used 2 second long windows and plotted only values where the average cross correlation between seismograms was greater than 0.8.

Figure 10 displays the slowness results obtained for the fault-parallel velocity at site 1. Zero back azimuth corresponds to the direction perpendicular to the San Andreas fault (N28°E). The first half-cycle in the seismogram is the direct S-wave (S1) and has an apparent velocity of about 3.5 km/sec. The back azimuth of the initial arrivals is about 300°, corresponding to the direction to the sub-event at the northwest end of the fault. Because of directivity, this sub-event produces the dominant ground
motion in most of the basin. Between 10 and 20 seconds, the seismogram consists of large phases which have been reflected at the free surface and the alluvium-basement interface. The peak velocity occurs at 14 sec and is caused by the phase that has been reflected twice between the free surface and the alluvium-basement interface (phase A). This is the orange wavefront that extends diagonally across the middle of the basin in Figure 9c. The array analysis (Figure 10) indicates that the large arrivals between 10 and 20 sec have apparent velocities between 0.9 and 2.0 km/sec, significantly slower than the direct S-wave. This slow velocity is caused by reflection at the dipping alluvium-bedrock interface (see Figure 4). There are two windows in this time period (10-20 sec) that have faster apparent velocities between 3 and 4 km/sec. The slowness estimates for these windows may be less reliable, because the windows do not contain portions of the waveform where the slope changes markedly.

At 20 sec, the back azimuth abruptly shifts to about 120° (Figure 10). This arrival has been reflected from the right edge of the basin. This is one of the dark blue wavefronts (phase C) stretching diagonally across the right half of the basin in Figure 9d. The apparent velocity of this arrival is about 2 km/sec, indicating it is a surface wave. The windows after 20 sec have a variety of back azimuths (Figure 10), with many arrivals coming from the southern edge of the basin. The apparent velocities of most of these arrivals range from 1 to 2 km/sec. Therefore, these later arrivals are surface waves travelling in various directions across the valley.

Figure 11 shows the synthetic seismograms at San Bernardino with and without anelastic attenuation (Q=50) in the sediments. Determining the appropriate $t^*$ at any given time is problematic because of the finite duration of the source. We do not know exactly how much time each arrival in the synthetic seismograms has travelled within the alluvium. I started the attenuation operator at 8 sec into the record so that the travel time in the alluvium was assumed to be the arrival time minus 8 sec. The 8 sec is the travel time of the direct S-wave from the sub-event on the left end of the fault. This procedure may over-attenuate energy radiated from the sub-event on the right end of the fault.

The largest horizontal motions at San Bernardino occur early in the synthetics (Figure 11). This represents the direct S-wave and the reflections from the free surface and alluvium-bedrock interface. There is significant long-period energy 30-50 sec into the records. Examination of time slices reveals that these arrivals are surface waves from various azimuths.

The synthetics at the valley site with the maximum amplitude (site 4) is shown in Figure 12, along with those from a rock site (site 5) 8 km away. Site 4 is located close to the area of maximum velocity in Figure 9d (red area in right portion of basin). The peak motions on the horizontal components of site 4 are at 10 sec into the simulation. These large motions are primarily caused by the constructive interference between the trapped waves behind the direct S-wave radiated from the left end of the fault and S to
surface wave reflections from the right edge of the basin. The extremely low amplitudes at the rock site are striking in comparison with the velocities at the valley site 4. The pulse on the N62°W seismogram for site 5 represents the radiation from the sub-event on the right end of the fault. The amplitude is largest on the component parallel to the fault because of the radiation pattern of the earthquake.

I calculated the maximum horizontal vector velocity found from the synthetics for each grid point on the surface. The horizontal vector amplitude at each time step is the square root of the sum of the squares of the two horizontal components. Figure 13 is a contour plot of the maximum horizontal velocities from the 3-D simulation for the rupture propagating from left to right (northwest to southeast). In general, basin sites exhibit much larger peak velocities than the rock sites. The largest ground velocity (about 180 cm/sec) occurs in the southeast portion of the basin, close to the red area in Figure 9d. The largest velocities are above the northward-dipping border of the basin. The contours are elongated southwestward from both ends of the fault due to the two asperities at the ends of the faults and their radiation pattern.

Peak motions at any given location in the valley are dependent on the direction of rupture propagation. A second M6.5 simulation was done for a rupture starting starting at the right end of the fault and propagating to the left. The final slip is the same as in the previous example (see Figure 8). Figure 14 displays the contour plot of maximum horizontal vector velocity for the northwesterly propagating rupture. This plot differs significantly from that for the southeasterly-propagating rupture (Figure 13). The location of the maximum velocity has shifted to the left and reduced in amplitude, and is now located near the trough of the basin. The constructive interference between waves reflected from the edge of the basin and the trapped waves following the direct S-wave is not as strong for the northwesterly rupture propagation. For the southeasterly propagation, the S-wavefront hit the right-side (south) of the basin broadside, producing strong reflections back into the valley. This energy was particularly strong because of directivity of the source from the sub-event at the left end of the fault. For the northwesterly propagating rupture, the energy from the sub-event at the right end of the fault is not as strongly reflected at the left edge of the basin. Seismograms at site 4 for the two rupture directions are shown in Figure 15. The large motions on the horizontal components for the northwest hypocenter (top row) are not present for the synthetics for the southeast hypocenter.

Discussion

Safak and Frankel (1992) used the synthetics from one of the M6.5 simulations to calculate response spectra and building response, comparing them with those from a flat-layered model. In most cases, the response spectral values for the 3-D synthetics were two to three times as large as those from the flat layered models, for periods of
1-4 sec. The synthetics were input into the base of a 10-story building. The maximum drift, displacement, and shear strain for the building were substantially larger for the synthetics from the 3-D case than those derived from flat layers. This initial study indicated that it is necessary to use synthetics from 3-D models to adequately assess the performance of tall buildings in sedimentary basins.

The results of this paper indicate that we must consider a range of rupture scenarios with varying rupture directivity and asperity locations when constructing maps of various ground motion parameters such as Figures 13 and 14 for future earthquakes. It is also likely that the generation of surface waves in the basin will be sensitive to the depth of the earthquake source.

While the present paper illuminates many aspects of wave propagation in 3-D sedimentary basins, its application is limited by the lack of detailed information about the velocities, depths, and Q values of the alluvium and underlying rock of the San Bernardino basin. Gathering these data is expensive. However, it is probably cost effective based on the sensitivity of tall buildings and highway overpasses to surface waves in the basins. As more recordings of earthquakes become available for basin sites, the predictions of the 3-D simulations can be tested and the basin models refined.

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References


Figure 1. Map of San Bernardino Valley corresponding to area used in simulations. Shaded areas represent exposed bedrock, unshaded areas are alluvium. Crosses are depths to bedrock (in meters) from well or refraction data. San Andreas fault forms the northeast edge of the valley. Contours represent the depth to bedrock used in the simulations. Contour interval is 100m. Map at right shows location of San Bernardino Valley.
Figure 2: Map showing location of epicenter of M5 earthquake (solid circle) and rupture extent of M6.5 earthquake used in simulations. Triangles are site locations referred to in text.
Figure 3. (top) Synthetic velocity seismograms at site 1, from 3-D simulation for M5 earthquake. (middle) Reflectivity synthetic seismogram from model with horizontal layers. (bottom) Synthetic for 2-D finite difference simulation. Peak amplitude is in cm/sec. For each figure in this paper, all seismograms are plotted on same scale.
Figure 4. Schematic of raypaths for an S-wave impinging on an alluvial basin with a 
dipping bedrock interface.
Figure 5. Synthetic seismograms at San Bernardino for 3-D (top), 1-D (middle), and 2-D (bottom) models, for M5 earthquake. All seismograms plotted with the same amplitude scale. Peak velocity given in cm/sec.
Figure 6. Synthetic seismograms at San Bernardino for 3-D simulation of M5 earthquake for cases of no anelastic attenuation (top), Q=50 in the alluvium (middle), and Q=25 in the alluvium (bottom).
Figure 7. Synthetic seismograms at site 2 (top) and site 3 (bottom) from the M5 simulation. For site 2, a Q of 50 was used for the alluvium.
Figure 8. Final slip (m) plotted as a function of position on the fault for the M6.5 simulation. The fault is 30 km in length by 3 km in depth extent.
Figure 9a-d (on 4 following pages). Snapshots of fault-parallel, horizontal velocity for the M6.5 simulation for four time steps. Each time step has three panels. Top panel shows slip velocity on fault plane. For this panel, downward represents increasing depth. Middle panel is a map view of ground velocity on the surface. Bottom panel is the velocity for a vertical cross section along the red line in middle panel (downward is increasing depth). The contact between alluvium and bedrock is marked by the orange lines, as is the location of the San Jacinto fault (see Figure 1 for reference map). Positive values (yellow, orange) indicate leftward particle velocity, negative values (dark blue, red) are rightward particle velocities. Red and orange indicate largest absolute velocities, light blue represents zero velocity. See text for description of phases S1, S2, A, B, and C.
fig. 9d 16.8 sec
Figure 10. Results from moving window determination of vector slowness from small array at site 1, for M6.5 simulation (see text). Top shows one of nine synthetic velocity seismograms (fault-parallel component) used in analysis. Back azimuth is measured in degrees clockwise from the perpendicular (N28°E) to the strike of the San Andreas fault. Backazimuth to sub-event at northwest end of fault is about 300°. Labelling of phases corresponds to that in Figure 9.
Figure 11. Velocity synthetics at San Bernardino for M6.5 simulation, without anelastic attenuation (top) and with Q=50 in alluvium (bottom). N62°W motion is horizontal component parallel to strike of fault. N28°E motion is component perpendicular to fault. UP is vertical motion.
Figure 12. Velocity synthetics at sites 4 and 5 for M6.5 simulation.
Figure 13. Contour plot of maximum horizontal vector velocity for M6.5 simulation, with rupture propagating from northwest to southeast. Contours labeled in cm/sec, contour interval is 20 cm/sec. Thick lines indicate borders of basin.
Figure 14. Contour plot of maximum horizontal vector velocity for M6.5 simulation, cm/sec.
Figure 15. Velocity synthetics at site 4 in valley for earthquakes with different directions of rupture propagation, northwest to southeast (top) and southeast to northwest (bottom).
RECENT SEISMOLOGICAL INSIGHTS INTO THE SPATIAL VARIATION OF EARTHQUAKE GROUND MOTIONS

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U.S. Geological Survey

ABSTRACT

This paper reviews recent seismological observations relevant to the determination of the spatial variation of earthquake ground motions over distances from 5 m to 1 km and in the frequency band from 0.1 to 50 Hz. Only elastic variations in ground motions are discussed, so the effects of landslides, liquefaction, lateral spreading, and surface faulting are not reviewed. Observations from the 1979 Imperial Valley, 1989 Loma Prieta, and 1992 Landers, California, earthquakes and their aftershocks are presented. Spatial variations in ground motion are quantified in terms of either the coherence of motions at different points, the ground strains caused by ground motions, or the apparent seismic wave speeds along the ground. The main causes of spatial variations of ground motion are 1) the wave passage effect, which is well understood seismologically, 2) the effect of the free-surface boundary condition, including the presence of surface waves, which have been observed in recent earthquakes, and 3) the effects of horizontal variations of geologic structure, such as the introduction of spatially variable delays in upward travelling wave fronts and spatially variable site effects. These are still not well quantified. Although it might be expected that earthquake source finiteness might affect the spatial variations of ground motions, most recent seismological work indicates that this effect is minimal. Specific examples of each of the above factors are given.

INTRODUCTION

It is an opportune time to review recent seismological work relevant to the spatial variation of earthquake ground motions because the 1989 Loma Prieta and 1992 Landers, California, earthquakes and their aftershocks provided considerable new data recorded on dense arrays of seismometers (arrays having interstation spacings from 25 m to 1 km). This paper will summarize the new seismic data and relate it to previous work on the spatial variations of ground motion. Frequencies between 0.1 and 50 Hz and interstation spacings of 5 m to 1 km will be treated in this paper, and I will concentrate exclusively on elastic variations in ground motions, i.e. variations in ground motions that are not caused by permanent inelastic deformations of the ground. Consequently the effects of landslides, lateral spreading, liquefaction, and slippage across faults will not be considered. I will not indulge in the mathematical details of the subject much; the appropriate papers where the precise details, definitions, and derivations can be found will be cited as necessary.

QUANTIFICATION OF SPATIAL VARIATION OF GROUND MOTIONS

Given a set of closely spaced recordings of ground motion time series, the differences between the ground motions are usually characterized either in terms of the relative strains between the recording points (for example, O'Rourke and others, 1984, or Spudich and
Cranswick, 1984a) or in terms of the coherence between the motions at the different points (defined, for example, in Abrahamson and others, 1991), or in terms of amplitude spectral ratios between observation sites (for example, Schneider and Abrahamson, 1992). The concepts of ground strain and spectral ratios are straightforward and clearly defined in the references cited above, so there is no need to define them further here. However, the use of coherence to describe ground motion variations is less obvious and is considerably more complicated.

**Coherence**

In this paper the term *coherence* is loosely used to refer to any of a number of related spectral-domain measures of the similarity of a group of time series. In order to clarify subsequent discussions, it is necessary to define some of these terms more specifically. The discussion below follows the definitions of Abrahamson and others (1991). If \( u_1(t) \) is the ground displacement at point \( x_1 \) at time \( t \), and if \( U_1(f) \) is the Fourier transform of \( u_1(t) \), where \( f \) is frequency, then the *cross-spectrum* of \( U_1(f) \) and \( U_2(f) \) is \( S_{12}(f) \), which is defined to be the expected value of the product \( U_1(f)U_2(f) \). The *auto-spectrum* of \( U_1(f) \) is \( S_{11}(f) \), which is defined to be the expected value of the product of \( U_1 \) with itself, \( U_1(f)U_1(f) \). The *coherency* of the motions at points \( x_1 \) and \( x_2 \) is a complex number \( \gamma_{12}(f) \) defined by

\[
\gamma_{12}(f) = \frac{S_{12}(f)}{[S_{11}(f)S_{22}(f)]^{1/2}}
\]

(1)

(see also Bendat and Piersol, 1993, pp. 54-55). Rigorously speaking, *coherence* \( \gamma^2 \) is a real number defined to be the square of the magnitude of the coherency. Possible values for both the magnitude of the coherency and the coherence lie in the range 0 to 1.

Coherence and coherency are used to quantify whether a set of signals are related by a linear transfer function. If only a single realization of the two spectra \( U_1(f) \) and \( U_2(f) \) is available, then it is always possible (if one of the spectra is not zero at some frequency) to find a linear transfer function that relates the two spectra, and direct substitution of \( U_1(f) \) and \( U_2(f) \) into the definitions of \( S_{12} \) and into (1) verifies that the coherency is unity in this case, regardless of what \( U_1 \) and \( U_2 \) are. However, as mentioned earlier, \( S_{12} \) is actually an expected value, and to determine the expected value of the spectral product, the product is usually averaged over a band of frequencies (e.g. Abrahamson and others, 1991) or it is averaged over several time and frequency windows (e.g. Menke and others, 1990) or averaged with more exotic windowing (Vernon and others, 1991, average over time windows using prolate spheroidal wave functions as taper functions). In the case of many signals, a single linear transfer function cannot usually be found that relates all the different realizations of \( U_1 \) to all the different realizations of \( U_2 \), and the resulting coherency has a magnitude less than 1.

Coherence and coherency are useful for several reasons. First, as will be discussed later, they can be used to quantify the spatial variations of ground motion caused both by the wave passage effect and by other random variations of ground motion unrelated to the wave passage effect. In addition, if coherence curves are available as functions of spatial separation and frequency, the cross-spectrum of ground motions at two different locations can be derived or estimated by multiplying the coherence and the autospectrum of ground motion at either of the two points (e.g. Der Kiureghian and Neuenhofer, 1992, equation 20). Autospectra of ground
motions are easily obtained from single-station ground motion records or from theoretical models of ground motions. Ground motion time series consistent with a given coherence function can be generated using the methods of Hao and others (1989) or Abrahamson (1993).

While coherence is a useful measure, there are important amplitude effects it is not sensitive to. Ratios of ground motion spectra from nearby sites are better suited to quantify spatial variations in the amplitudes of ground motions (for example, the $\sigma$ factor of Abrahamson and others, 1991, and Schneider and Abrahamson, 1992). It is necessary to consider spectral ratios as well as coherence because a measured coherence is insensitive to amplitude differences between signals. For example, if the ground displacement at a site is $u(t)$ and the displacement at a nearby site is $4u(t)$, the coherence between these motions is 1, exactly the same value that would be obtained if the motion at both sites were $u(t)$.

CAUSES AND EXAMPLES OF SPATIAL VARIATIONS OF GROUND MOTIONS

In this section I will discuss the causes of spatial variations of ground motions starting with a geologically unrealistic picture of the Earth as a uniform medium of infinite extent, and I will progressively add aspects of realistic geology and present examples of how these factors affect the spatial variations of ground motions.

Wave passage effect

If we consider a point seismic source in an infinite medium, the particle motions at two adjacent points differ from each other primarily because of the wave passage effect, i.e. because of the difference of wave arrival time between one observation point and another. (Here I neglect spatial variations in wave amplitude caused by source radiation patterns, $1/R$ amplitude decay with propagation distance, and effects caused by near-field terms because these are all small effects compared to others that occur when we introduce more realistic geologies.)

Regarding estimation of ground strains, if we assume the waves from the source are planar (while they are usually not planar, the actual deviations from planarity are generally caused by geological heterogeneities, discussed later), then the expressions for the strains in an infinite medium caused by plane waves are elementary and are given by Spudich and Cranswick (1984a), as well as by many prior authors. At this point the concept of the apparent velocity of the waves becomes important. I will belabor this elementary point primarily because seismologists have made many measurements of apparent velocities of body and surface waves in recent earthquakes, and selection of the appropriate apparent velocities is the focus of articles in the earthquake engineering literature (for example, O'Rourke and others, 1982; O'Rourke and others, 1984). If $x$ is the distance along an axis in space, and if $u(x,t)$ is the particle displacement at position $x$ and time $t$, then the spatial derivative of $u$ is

$$\frac{du}{dx} = \frac{-d(t)}{c},$$

(2)

where $c$ is the apparent velocity of the wave front along the $x$ axis. This expression shows that for nondispersive body waves, the spatial gradients of displacement are proportional to particle (or ground) velocity, which is readily observed. For this reason considerable effort has gone into determining typical values for the apparent velocity $c$ of strong ground motions. Expression
(2) is valid for body waves, which are nondispersive except in highly attenuating materials. As we will see later, a very similar expression for strain, in which \( c \) is replaced by \( c(f) \), which is a frequency-dependent apparent velocity, works very well for surface waves. (A note on seismological terminology: the apparent velocity of surface waves, which is the speed of propagation along the ground surface of the peaks, troughs, and zeros of the surface wave packet, is usually called the phase velocity, to distinguish it from the speed at which the wave packet envelope advances, the group velocity, as in Aki and Richards, 1980, p. 264. In seismology, the slowness is \( 1/(\text{apparent velocity}) \), and is a very convenient number because it is linearly related to wave arrival time differences, and unlike apparent velocity it never becomes infinite. All these terms, i.e. apparent velocity, phase velocity, and slowness, are usually understood to refer to propagation along the ground surface in the direction of wave advance, i.e. in the direction of the projection of the wave front normal onto the ground surface.)

Regarding coherence, the effect of the arrival time differences caused by the wave passage effect is to change the phase of the complex coherency \( \gamma \) without altering its magnitude (Abrahamson and others, 1991; Der Kiureghian, 1993; Bendat and Piersol, 1993, pp. 170-171). If \( \gamma_{12}(f) \) is the coherency of motions at \( x_1 \) and \( x_2 \), and if \( d_{12} \) is the distance between \( x_1 \) and \( x_2 \), then the coherency of a single propagating wave can be expressed in the form

\[
\gamma_{12}(f) = |\gamma_{12}(f)| \exp[-2\pi id_{12}/c(f)].
\] (3)

In (3) the effect of plane wave propagation on coherency is entirely represented in the exponential term, and the magnitude of the coherency (called the lagged coherency by Abrahamson and others, 1991) includes other factors, to be discussed below, that cause the ground motions to differ between points \( x_1 \) and \( x_2 \). Consequently, the variation in coherence caused only by the wave passage effect is straightforward to estimate theoretically, at least when only a single plane wave is present (often a good assumption for strong motions).

Effects of the free surface and a seismic velocity structure that varies with depth only

If we introduce a free surface and a seismic velocity structure that varies as a function of depth only, two important new phenomena occur that affect spatial variations of particle motions. First, the boundary condition that tractions are zero on the free surface causes vertical derivatives of particle displacement to go to zero at the surface, which would be an important factor for buried vertically oriented structures. Although they do not calculate vertical derivatives explicitly, O'Rourke and others (1982) show a proper treatment of the free surface (see also Aki and Richards, 1980, pp. 135-144). On the other hand, Spudich and Cranswick (1984a), who calculated particle displacement gradients caused by the 1979 Imperial Valley earthquake, neglected this factor and severely overestimated derivatives of displacement with respect to depth (their horizontal derivatives are correct). If we allow the \( x \) axis to be horizontal, then expressions for horizontal derivatives of body waves, like (2) above, are unchanged by the presence of a free surface. The second effect of a free surface and a velocity structure that varies with depth is that dispersive surface waves can propagate along the free surface. A relation very similar to (2) can be shown to be true for surface waves, except that the apparent velocity \( c \) becomes a function of frequency (Mikumo and Aki, 1964; Spudich and others, 1994). The appearance of surface waves is important for several reasons. First, surface waves often dominate the long duration strong ground motions observed in alluvial basins (e.g.
the Los Angeles basin after the 1971 San Fernando earthquake, Hanks, 1975; Liu and Heaton, 1984). More examples will be discussed below. Second, the apparent velocities of surface waves are generally smaller than those of body waves, leading to higher strains. O'Rourke and others (1984) give a good discussion of many of these concepts. However, they also assert the existence of an effective propagation velocity of surface waves that varies with station separation (see their Figure 4), and many seismologists would take issue with this assertion.

Effects of horizontal variations of geological structure

The general picture of crustal structure emerging from seismology is that there are strong horizontal variations in crustal velocity and density, but they are concentrated near the Earth's surface and decay with depth. These horizontal variations occur on all scale lengths from kilometers to meters. From study of the coherence of 5-30 Hz seismic waves recorded by an array on a hard rock site, Menke and others (1990) concluded that the observed coherence is consistent with geological heterogeneities concentrated in the top few kilometers of the crust. Vernon and others (1991) observed that the coherence of motions from sensors at 150m and 300m depth was higher than the coherence of motions recorded at the ground surface on sensors having comparable spacing to the borehole sensors, implying concentration of geologic heterogeneities in the top 100m of the crust. The existence of near-surface heterogeneity is relevant to the engineering question of spatial variations of ground motions in three ways.

First, horizontal variations in material velocity cause spatially variable advances and delays in an upward travelling seismic wave front, adding additional phase shifts to those caused by the wave passage effect and causing additional spatial variations of ground motions. Spatial variations caused by arrival time perturbations may be substantial and are usually neglected in strong motion studies. An excellent example of the importance of arrival time deviations is given by Harichandran (1988, his figures 4 and 5), who examined SMART I array data (Bolt and others, 1982) and showed that local deviations of S-wave arrival times from those predicted for a plane wave arrival were as large as 0.2s and completely altered the direction of wave passage for some locations within the array. Der Kiureghian (1993) has included the effects of arrival time perturbations in a coherence model.

It is difficult based on strong motion array data to estimate the usual magnitude of these sorts of arrival time perturbations because their measurement requires that the relative timing of records from a strong motion array be known very accurately, perhaps to 0.01s. Often this is not known because the strong motion stations do not have clocks (for example, the 1971 San Fernando earthquake, O'Rourke and others, 1984) or the clocks fail (for example, the 1979 Imperial Valley earthquake, Spudich and Cranswick, 1984a, 1984b), or the clock drifts are not known accurately (for example, the 1992 Landers, California, earthquake, Hough and others, 1993; Hough, 1994). In each of these cases, the relative array timing was artificially set by identifying a seismic pulse on the accelerograms, assuming a known apparent velocity for the pulse, and deriving the array timing from this assumption. This procedure removes all arrival time perturbations from the ground motions, which has the effect of biasing estimates of ground strain downward. The geophysical exploration industry could provide an abundance of data on the spatial variation of arrival time perturbations, since they collect large amounts of data from closely spaced sensors in a variety of geologies. These time perturbations are called 'statics'.
or ‘static corrections’ in that literature, and an example of the spatial variation of these static corrections can be seen in Berni and Roever (1989).

Second, horizontal variations of seismic velocity and density cause corresponding variations in ground motion amplitude simply because of the variation of seismic impedance (product of density and seismic velocity, see Aki and Richards, 1980, p. 137) at the Earth’s surface. The inverse correlation of ground motion amplitude with surface shear wave velocity has been shown by Borcherdt (1992), Midorikawa and Sakugawa (1993), and Boore and Joyner (this volume). This mechanism can operate over short horizontal distances, and it has also been shown to operate as a function of depth by Archuleta and others (1992). Because ground motions are larger in slower velocity material, it has been commonly observed in exploration seismology that larger motions tend to be correlated with delayed waves.

Third, lateral variations of velocity and density, if large enough, cause reflections and scattering of incident upcoming waves. These sources of scattered waves may lie inside or outside of the region of interest (e.g. the foundation of a large building), and their effect will be to reduce the coherence of ground motions (Bendat and Piersol, 1993, pp. 183-185). There are many observations from seismology of scattered waves impinging on a region of interest from outside the region, and these will be discussed later. When the scattered waves are generated by heterogeneities within the region of interest, their effects are usually characterized as ‘site effects.’ Ground motion amplitudes can vary by factors of two over distances of 20m or more (Schneider and others, 1992, their figures 5 and 6) depending on the frequency band. King and Tucker (1984) observed a factor of five variation of ground velocity over a distance of 200m related to variations of sediment thickness in an alluvial valley. During the 1979 M6.9 Imperial Valley earthquake, peak accelerations varied by a factor of 2 over a horizontal distance of 200m along the El Centro Differential Array (Bycroft, 1980). A similar factor of two variation in peak acceleration and 1.5 in ground displacement was observed by Hough (1994) over a distance of 500m on the Morongo Valley array in the 1992 Landers earthquake. In both these cases, the array instruments were all situated on similar deposits of alluvium. A geological situation that might be common is an alluvial layer overlying a rough basement surface. In theoretical studies of ground motions in this type of structure, Hill and Levander (1984) and Levander and Hill (1985) showed that upcoming body waves in the basement convert into surface waves in the alluvium after hitting the irregular boundary. Because the phase velocities of these surface wave would be relevant to the wave propagation effect, it is significant that the phase velocities of the surface waves in the alluvium layer were those that would be expected for an average smooth basement. Hence, it may not be necessary to know the detailed structure of the alluvium-basement contact to predict the phase velocities of surface waves that might be generated in the basin.

Fourth, topographic relief is a manifestation of geologic heterogeneity that may strongly affect coherence of ground motions. A good review of the effect of topography on ground motions is given by Geli and others (1988). The effect of the sloping ground surface is to reflect upcoming waves having high apparent velocities into more horizontally travelling paths having much lower apparent velocities. This can cause the ground motions to have short horizontal wavelengths, and it also causes considerable incoherence of ground motions. Hartzell and others (1994) show important observations of these effects in dense array recordings of aftershocks of the Loma Prieta earthquake. Schneider and others (1992) show an example of
a dense array on a rock site in which topography may be the cause of low coherence (discussed below).

As noted above, horizontal variations in seismic velocity and density cause spatial variations of both phase and amplitude of ground motions, and these variations are in addition to the phase variations caused by the wave passage effect. For engineering purposes, these variations have been characterized in two ways. First, these effects affect the lagged coherency, i.e. the magnitude of the coherency, \(|\gamma(f)|\). To account for these effects on coherency, a number of investigators including Luco and Wong (1986) parameterized the lagged coherency in terms of frequency \(f\) and separation distance \(d\), \(|\gamma(f)| = \exp[-\alpha(fd)^n]\), where \(n\) is usually 2 and \(\alpha\) is an empirically determined constant. Note that in this formulation, the decay of lagged coherency with frequency is equal to its decay with separation. Numerous other parameterizations of lagged coherency have been suggested (see Schneider and Abrahamson, 1992; see also Harichandran, 1988, for a review). However, as noted earlier, the coherency measure is insensitive to amplitude variations, so Abrahamson and others (1991) and Schneider and Abrahamson (1992) have additionally used a measure \(\sigma(f,d)\) to quantify observed amplitude variations.

**Effects of earthquake source finiteness**

In principle the spatial extent of a nearby rupturing fault might be expected to reduce the coherence of ground motions owing to the multipath propagation of seismic waves, but the small amount of observational data does not indicate that the spatial extent has a strong effect, and theoretical models of earthquake rupture also indirectly support the small effect of source finiteness on coherence of ground motions. Ideally, the effect of source finiteness could be determined by examining ground motions caused by a large earthquake and its foreshocks or aftershocks recorded on a seismic array. While dense seismic array data exist to do such a comparison (e.g. the 1986 Hualien earthquake and its foreshock, Goldstein and Archuleta, 1991; the 1979 Imperial Valley earthquake and its large aftershock, Spudich and Cranswick, 1984b; the 1992 Landers earthquake and its foreshocks, Hough, 1994), only Somerville and others (1988) and Abrahamson and others (1991) specifically examined the dependence of coherence on earthquake magnitude. The former authors find a magnitude dependence for the Imperial Valley events, while the latter authors, examining a larger data set, do not. Because few of Abrahamson and others’ data were observed particularly close to a large earthquake, their observed independence of coherence on magnitude provides only a weak indication that source finiteness is not an important effect. However, other circumstantial evidence suggests that source finiteness may not affect coherence strongly. Numerous studies of strong ground motion support the idea that most of the waves radiated by large earthquake originate in a spatially compact region that travels with the earthquake rupture front (Heaton, 1990). Studies of dense seismic array data from large earthquakes tend to support this idea, as will be demonstrated below. Consequently, for earthquakes having unilateral rupture propagation, which may represent the majority of earthquakes, the portion of the fault that is radiating at any particular instant of time may be only a small fraction of the total ultimate rupture area. For bilateral earthquakes, such as the 1989 Loma Prieta earthquake (e.g. Beroza, 1991; Steidl and others, 1991; Wald and others, 1991) there are probably two relatively compact sources of seismic energy radiating simultaneously. In this case, multipathing would have to be considered, but probably only from two moving sources of energy. Unfortunately, no dense seismic array recordings of the Loma
Prieta earthquake exist, so the effect of bilateral rupture propagation, if any, on ground motion coherence is still unmeasured.

LESSONS FROM DENSE SEISMIC ARRAY RECORDINGS OF RECENT EARTHQUAKES

In this section I will try to provide observed examples of many of the phenomena discussed above, concentrating on observations made on dense seismic arrays. Observations will be emphasized that may be relevant to the estimation of ground strains and coherence, these observations primarily being identification of the types of waves present in strong ground motions (e.g. body waves, surface waves), observations of the apparent velocities of these waves, and estimations of strain and coherence made from these observations. Rather than surveying the entire literature on this subject, I will concentrate on recent results or on results buried in the seismological literature that may not be well known in the engineering world.

The problem of seismic array timing

A common denominator of most of the studies cited below is that information on true array instrument timing was not used, either because the true instrumental clocks were inaccurate or the investigator's purpose did not require use of the actual array timing. In most of these studies the investigator chose a particular seismic pulse on the records (usually the initial P or S pulse from the earthquake hypocenter), assumed a desired apparent velocity for this pulse, and adjusted the instrument times accordingly so that the pulse appeared to propagate across the array at the assumed apparent velocity. Usually the assumed apparent velocity was a theoretical value derived from simple geometric optics considerations and Snell's law, and in all these studies no data inconsistent with the assumption were noted. Consequently, the assumed apparent velocities are probably accurate to better than a factor of 2, but there is still some uncertainty owing to the assumption of array timing.

Survey of coherences by Schneider and Abrahamson (1992)

Schneider and Abrahamson (1992), in a very comprehensive study of existing dense array data sets, calculated coherences using data from numerous dense seismic array data sets and found that a single functional relationship between frequency, instrument separation, coherence, and amplitude variation failed to account for the observed correlations of these quantities (Figure 1). Although Schneider and Abrahamson were unable to identify conclusively the factor causing the variations they observed, the causative factor appeared to be related to site geology and not related to earthquake magnitude. Figure 1 shows their compilation of coherence as a function of frequency for two types of geology and two ranges of station spacings, 15-30m and 50-80m. The solid and dashed lines show an empirical relation they had previously determined from the LSST array in Taiwan (Abrahamson and others, 1991). The main observation is that in the results for rock sites, two of the arrays, the USGS Parkfield array (UPSAR, Fletcher and others, 1992) and the ZAYA array (Bonamassa and others, 1991) show considerably different coherence from all of the other arrays, most of which agree fairly well with the LSST empirical relations. The data for the UPSAR and ZAYA arrays were for earthquakes in the magnitude range of 2.2 to 3.5, whereas the magnitude range recorded by all the other arrays was 2.0 to 7.8. Consequently, it is quite clear that some unknown factor related to station geology and/or
topography can have a much larger effect on coherence than earthquake size. This observation tends to support the assertion made earlier that earthquake source finiteness in general does not seem to affect coherence much; a specific example in Figure 1 is provided by the data points for the Imperial Valley (El Centro) Differential Array (squares), which was located 5.6 km from the surface trace of a rupture that extended more than 40 km (Figure 2). Although these data points include the contributions of both the 1979 main shock (M 6.9) and its largest aftershock (M 5.1), if we assume that the aftershock had coherences comparable with the LSST coherence curves, then the main shock must also have had comparable coherences to produce the plotted average values.

Body waves, surface waves, or both?

Most recent seismological data indicate that in alluvial basins strong ground motions can consist of both body waves and surface waves, even at distances that are nearer to the earthquake source than the usual rule-of-thumb (which is that surface waves are only observed at epicentral distances more than 2-6 times the earthquake depth; see Kamiyama and others, 1992, p. 9-4). The main mechanism for surface wave generation near the source is the conversion at the boundary of the alluvial basin of S body waves to surface waves that propagate in the alluvium. This is an example of scattered waves generated by local spatial variations of geology, discussed earlier. Examples of these locally generated surface waves are those observed in the Los Angeles and San Fernando basins during the 1971 San Fernando earthquake (Hanks, 1975; Liu and Heaton, 1984), in the Santa Clara valley generated by aftershocks of the 1989 Loma Prieta earthquake (Frankel and others, 1991; Frankel, this volume), in the Kanto plain generated by Tokyo-region earthquakes (Phillips and others, 1993), in the Kyoto basin (Horike, 1988), in the San Bernardino valley generated by aftershocks of the Landers earthquake (Frankel, 1994; Frankel, this volume), and in the Morongo valley generated by the 1992 Landers earthquake and its aftershocks (Hough and others, 1993; Hough, 1994). Although there are observations of large earthquakes in alluvial basins during which no surface waves are detected (e.g. the 1979 Imperial Valley earthquake, Spudich and Cranswick, 1984b), the bulk of the seismological evidence indicates that it is quite common for surface waves to be generated at the edges of alluvial basins and propagate in the basins. The surface waves observed in the Kyoto basin, Santa Clara valley, San Bernardino valley, and the Morongo valley have apparent velocities as low as 1.0 km/s, which is considerably slower than the apparent velocities associated with S body waves.

For rock sites within 50 km of a large earthquake there is little data indicating that surface waves are a substantial contributor. At rock sites at larger distances, however, surface waves generated at the seismic source can sometimes be observed at periods of 5-10s (Campillo and Archuleta, 1993; Spudich and others, 1994). Specific observations based on array studies follow.

The 1979 Imperial Valley earthquake, M 6.9

This earthquake was observed at very close distance on the El Centro Differential array (Figure 2) and has been well studied to determine soil strains, coherence of ground motions, and apparent velocities of seismic waves. From the standpoints of this discussion, the main results are that the strong motions observed at the array consisted almost entirely of P and S body waves.
waves, most of the observed ground motions originated in a spatially compact region of the fault (so there was no obvious loss of coherence from the finite extent of the source), apparent velocities of P waves were between 5 and 10 km/s (stippled region, Figure 3), apparent velocities of the S waves were between 3 and 5 km/s (stippled region, Figure 4), and peak horizontal soil strains were about $2.2 \times 10^4$ (Bycroft, 1980; Smith, Ehrenberg, and Hernandez, 1982; Spudich and Cranswick, 1984a, 1984b). Figure 2 shows the layout of the El Centro Differential array. (Note that the array was linear, which caused problems in the analyses because only the component of apparent velocity in the north-south direction could be measured; arrays of this type should be two-dimensional.) Figure 3 shows the results of cross-correlation analysis performed by Spudich and Cranswick (1984b) for vertical accelerations (primarily P waves) and Figure 4 shows the same analysis applied to the horizontal accelerations (S waves). The main observation to be made in these two figures is that during the strongest part of the shaking, the north-south component of slowness of the waves ($1/\text{apparent velocity along the north-south direction}$) varied rather continuously from negative to positive values, indicating that the source of the waves moved from a position somewhere south of the array to a position somewhere north of the array. Spudich and Cranswick (1984b) identified this moving source as the earthquake rupture front, and they showed that its movement as inferred from the apparent velocities of the waves was consistent with its movement determined from modelling of strong ground motions (e.g. Hartzell and Heaton, 1983; Archuleta, 1984). The significance of this for engineering is that it indicates that the strong ground motions were well approximated by a single plane wave traversing the array, with the direction of plane wave propagation changing gradually during the shaking in a manner consistent with expected rupture velocities. In addition, the source of the waves was spatially compact, so that there was no obvious loss of coherence from source finiteness.

The M6.3 event of 1/29/81 and the M7.0 Hualien event of 11/14/86 observed on the SMART-I array in Taiwan

Observations of ground motions from the M6.3 event of 1/29/81 and the M7.0 Hualien event of 11/14/86 observed on the SMART-I array in Taiwan by Goldstein and Archuleta (1991) at distances of about 30km and 90km respectively showed that the ground motions in the 2-10 Hz band consisted primarily of body waves (although at 0.5 Hz surface waves may have been present), that the actual part of the source that was radiating at any particular instant was smaller than the ultimate fault size, and that for the 1981 event the apparent velocity of the P waves ranged between 7.10-8.38 km/s and the apparent velocity of the S waves ranged between 3.80 - 4.71 km/s (based upon a reasonable assumption of P-velocity/S-velocity ratio). Figure 5 shows the location of the SMART-I array and the 1981 and 1986 events. Figure 6 shows a subset of the vertical and horizontal component array data. Overlapping time windows of the vertical and horizontal motions were each analyzed to determine the spectrum of wave slownesses present in the window. A representative window is indicated at the bottom of the figure. From each window a slowness distribution was determined (see Goldstein and Archuleta for details), and these are shown for P waves in Figure 7. If the data consisted of a single plane wave propagating northwest at 10 km/s, it would plot as a delta function at $s_\tau = -0.07072$ s/km and $s_n = 0.07072$ s/km on this plot. The main observation to be made from this plot is that for none of the time windows does there appear to be more than a single resolvable slowness present in the data, supporting the idea that only a restricted part of the earthquake source is radiating at any instant. Consequently, the spatial extent of the earthquake source is not important in this
case, and there is probably little if any loss of coherence owing to source finiteness. The width of the peaks in Figure 7 is caused by the limited resolution of the array. As a final note on this work, although Goldstein and Archuleta (1991) do not analyze or discuss them, in Figure 6 there appear to be conspicuous waves having 2s period on the vertical components after about 12s. These may be Rayleigh surface waves, so it cannot be said that the ground motions observed from these earthquakes consist entirely of body waves.

The 1992 M7.3 Landers earthquake recorded on the Morongo Valley array

The M7.3 Landers earthquake and an immediate M5.6 aftershock, the Eureka Valley event, were observed on a 3-station array at about 20 km epicentral distance (Figure 8) by Hough and others (1993) and Hough (1994). The spacing between array elements was 300-500m. The measurements made by Hough are again consistent with others presented above, namely that the horizontal strong motions in both events are primarily caused by S waves propagating from a moving rupture, although there is evidence for late locally-generated surface waves observed during the Eureka Valley aftershock. Owing to the few stations in the array, the determination of apparent velocity and azimuth has limited resolution in this study.

Before considering the main shock motions, it is instructive to consider the M5.6 Eureka Valley aftershock. The aftershock example will show, as in other examples, that the initial strong motions are dominated by S body waves from a moving rupture, with possible later surface waves. All analysis was done on horizontal ground displacement records that were high-pass filtered above 0.2 Hz. Figure 9 shows the observed azimuth toward the source in the upper panel and the apparent velocity of propagation below. Each symbol indicates the apparent velocity or azimuth corresponding to a 2.5s long segment of the displacement records. In the upper panel the increase in azimuth from 5-8s is interpreted by Hough (1994) to correspond to direct S body waves from a rupture that propagates about 11km north to south along the Eureka Peak fault (Figure 2). During this time the corresponding apparent velocities are between 1.0 and 2.0 km/s. At about 10s the azimuth changes completely, and the apparent velocities assume a steady value of about 1.0 km/s. Hough interprets the low apparent velocities after 10s to correspond to surface waves locally generated in the Morongo Valley.

The results in Figure 10 for the main shock are consistent with those for the aftershock. In the upper panel, the dashed curve shows the azimuth variation to be expected from a smoothly propagating rupture that travels northward along the mapped faults from the main shock epicenter. The observed azimuths and apparent velocities are generally consistent with direct S body waves and locally generated surface waves, as was observed in the Eureka Valley aftershock. During the strong shaking the apparent velocities range between 1.0 and 2.0 km/s. It should be noted that the array timing was set assuming that the initial S wave arrivals have an apparent velocity of 1.8 km/s, which is a slower apparent velocity for S body waves than is usually observed. Hough states that the low body wave apparent velocities probably result from wave refraction caused by the dipping basement-alluvium contact beneath the seismic array.

The 1992 M7.3 Landers earthquake recorded on the UPSAR array

Ten stations of the UPSAR dense array (Figure 11; Fletcher and others, 1992) recorded the Landers earthquake at a distance of about 400km (Spudich and others, 1994). While the
frequency range and epicentral distance of their observation are not directly relevant to strong motion engineering problems, this example is included because it uses certain methodologies that may be useful for engineering problems. Spudich and others focused their work on determining the ground stresses, but their work is directly relevant to determination of the ground strains since the ground strains are related to the stresses by a simple linear scaling constant, the elastic modulus. In their work they were able to determine that the Landers ground displacements at UPSAR were dominated by source-generated surface waves, they were able to estimate the Fourier spectrum of the ground stresses directly from inversion of the interstation displacement differences, and they were further able to show that these stress spectra were very well approximated by a simple rule-of-thumb similar to (2) based on the ground velocity spectrum.

The question posed by Spudich and others (1994) was to ask how well the motions of the UPSAR array stations could be approximated by a spatially-uniform time-varying strain distorting the ground under the entire array, and by a time-varying rigid body rotation of the entire array. Because the ground displacements from Landers had primary frequencies of about 0.2s, the wavelengths of the waves were several km long, and consequently the array spanned only a fraction of a wavelength, so the assumption of a uniform strain and rotation of the array was valid. Figure 12 shows an example of a recorded ground motion. Note that the strongest motions last about 250s and the peak vertical displacement is about 1 cm. To perform the analysis, at every time step in the observed displacements, they found the best fitting uniform strain and rigid body rotation, yielding time series for these quantities. In Figure 12 it can be seen that the maximum strain had a value of about $1.0 \times 10^5$ about 160s into the record, and the maximum rigid body rotation of the array was about $6 \times 10^6$ radians at 130s into the record. During the strongest motions from 70-250s in the record, the misfit ratio of about 0.5 indicated that half of the total observed ground displacements were fit by the uniform strain and rigid rotation model (see Spudich and others, 1994, for details).

The work of Spudich and others (1994) also demonstrated that an expression very similar to (2) provided an excellent approximation to the actual ground stresses observed on the UPSAR array during the Landers earthquake. In the following I will drop all the miscellaneous scaling coefficients for simplicity; their equations (B3) and (B6) can be paraphrased as follows. Let $e(t)$ and $v(t)$ be the strain and ground velocity, respectively, as functions of time $t$, and let $E(f)$ and $V(f)$ be their Fourier transforms, then

$$E(f) \propto V(f)/c(f)$$

where $c(f)$ is the frequency-dependent phase velocity ‘dispersion curve’ of the Rayleigh or Love surface wave (Aki and Richards, 1980, pp. 260-265). The phase velocity $c(f)$ is the apparent velocity of a surface wave wavefront, exactly as $c$ in (2) is the apparent velocity of a body wave wavefront. However, for surface waves the phase velocity is a function of frequency, with lower frequencies having higher velocities. The Landers surface waves observed at UPSAR had phase velocities between 2 and 5 km/s.

Fourier spectra of ground strain (and stress) derived from interstation displacement differences compared surprisingly well with stress and strain spectra estimated from (4) based upon ground velocity records and theoretical phase velocity curves for the Parkfield geological structure. Figure 13 shows three pairs of Fourier amplitude spectra, corresponding to the three
nonzero elements of the stress tensor at the free surface, \( \tau_{rr} \), \( \tau_{rt} \), and \( \tau_{tt} \), where \( r \), \( t \),
and \( z \) are radial, azimuthal, and vertical unit vectors in a cylindrical coordinate system. In each
pair, the spectrum labelled 'observed' is the spectrum derived from interstation displacements,
and the curve labelled 'inferred' is the spectrum derived from \((4)\) and the indicated observed
ground velocity record. The agreement of the two methods for estimating stress (and strain) is
quite good, except in some cases for periods longer than 10\( \text{s} \), in which case the 'observed'
curves may be biased high because of noise.

This work suggests that the inference of ground stresses and strains from ground
velocities can be very accurate if reasonable phase velocity curves can be obtained. If the seismic
velocity structure of the site is known, calculation of phase velocities is straightforward using
standard routines such as those of Herrmann \((1985)\).

**HOW CAN THIS INFORMATION BE USED IN ENGINEERING APPLICATION?**

Summarizing the material above, recent seismological results indicate that the main
causes of spatial variation of ground motions are the wave passage effect and the effects of
horizontal variations of site response. The finite size of the earthquake source appears to cause
little degradation of the spatial coherence of ground motions, except possibly for bilaterally
rupturing faults, for which little data are available.

**Estimation of the wave passage effect**

The wave passage effect is fairly well understood seismologically. It is well
approximated by expressions like \((2)\) and \((4)\), which require estimates of apparent velocities.
Because strain is inversely proportional to apparent velocity, use of the lowest likely apparent
velocity gives a conservative estimate of ground strain. Surface waves usually have apparent
velocities lower than those of body waves, so it is necessary to consider whether they will be
present at a site. For sites in alluvial basins we have seen that it is quite common for surface
waves to be present in these basins regardless of epicentral distance. For rock sites that are
within about 30 km of the seismic sources, surface waves have not commonly been observed,
although they may be observed at greater distances. Consequently, it is reasonable to assume
at all alluvial sites that surface waves may be present. To determine the surface wave phase
velocity dispersion curves for a particular site in a heterogeneous geological structure, it is a
valid seismological approximation to calculate the dispersion curves using the velocity structure
directly under the site, ignoring horizontal variations of geologic structure \((\text{Woodhouse, 1974;}
\text{Tanimoto, 1990})\). These curves \( c(f) \) and an estimate of the expected ground velocity Fourier
spectrum \( V(f) \), can be used in expressions like \((4)\) to estimate the expected ground strain
spectrum for the wave passage effect.

**Estimation of the effects of lateral variation of site effects**

Unfortunately, recent seismological results only indicate that the effects of lateral
variation of site response are significant, and these results provide no good way to estimate them
theoretically. To estimate them it is necessary to fall back upon empirical observations such as
the lagged coherency and the \( \sigma(f,d) \) factors derived by Abrahamson and others \((1991)\) and
Schneider and Abrahamson \((1992)\).
Regarding the effects on coherency, as indicated earlier, a common parameterization of lagged coherency is of the form $|\gamma_{12}(f)| = \exp[-\alpha(fd)^n]$, where $n$ is usually 2 and $\alpha$ is an empirically determined constant. This form predicts that the lagged coherency will decay with distance as rapidly as it decays with frequency. However, most recent observational data indicate that the decay with frequency is faster than the decay with distance. While Menke and others (1990) find that an $\exp[-\alpha(fd)^1]$ form fits their data well, Vernon and others (1991), Abrahamson and others (1991), and Schneider and Abrahamson (1992), who review an extensive data set, find a more rapid decay of lagged coherency with frequency than with distance, at least for separations $d < 100$m.

As indicated earlier, lateral variations in ground motion amplitudes can be substantial. These have been measured using the $\sigma(f,h)$ factors of by Abrahamson and others (1991) and Schneider and Abrahamson (1992), but this type of analysis should be extended to much larger data sets.

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Soil Site Coherency

![Graph showing coherency for soil, 15-30 m and 50-80 m]

Rock Site Coherency

![Graph showing coherency for rock, 15-30 m and 50-80 m]

Figure 1. Comparison of observed coherences from several arrays (dots, triangles, squares, etc.) with empirical coherence functions (solid lines and dashed plus/minus one standard deviation curves) derived by Abrahamson and others (1991). Vertical axis is inverse hyperbolic tangent of the lagged coherency. Top row: results from arrays on soil. Bottom row: results from arrays on rock. Left and right columns show results for interstation separations of 15-30 m and 50-80 m, respectively. Note that most soil results agree fairly well with the empirical curves, but for rock arrays there is greater variation and lower coherence. Figure taken from Schneider and Abrahamson (1992).
Figure 2. Map of Imperial Valley showing the El Centro Differential array location and surface faulting caused by the 1979 earthquake. Inset shows the layout of digital accelerometers DA1 - DA6. Note that the array was only about 6 km from a fault rupture of about 40 km length. Figure from Spudich and Cranswick (1984a).
Figure 3. Moving window analysis of vertical accelerations (P waves) from the M 6.9 1979 Imperial Valley earthquake from stations DA1 and DA3 (see inset, Figure 2) of the El Centro Differential array. Width and weighting function of the moving window shown on the left side. The upper panel shows the peak value of correlation between stations DA1 and DA3 accelerations as a function of time. Middle panel shows DA1 vertical accelerogram for reference. Bottom panel shows the observed north-south component of slowness (1/apparent velocity) of the P waves as a function of time (solid line) and the inferred slowness in the direction of wave propagation (stippled region). Peak correlation is high for the first 9 s of the motions, indicating the duration of direct P waves from the source. During this period the n/s component of slowness goes from negative values (indicating northward propagating waves) to positive values (southward propagating waves). Adapted from Spudich and Cranswick (1984a,b).
Figure 4. Moving window analysis of north-south accelerations (S waves) from the M 6.9 1979 Imperial Valley earthquake from stations DA1 and DA3 (see inset, Figure 2) of the El Centro Differential array. Width and weighting function of the moving window shown on the left side. The upper panel shows the peak value of correlation between stations DA1 and DA3 accelerations as a function of time. Middle panel shows DA1 n/s accelerogram for reference. Bottom panel shows the observed north-south component of slowness (1/apparent velocity) of the S waves as a function of time (solid line) and the inferred slowness in the direction of wave propagation (stippled region). Peak correlation is high between about 5 and 10 s, indicating the duration of direct S waves from the source. During this period the n/s component of slowness goes from negative values (indicating northward propagating waves) to positive values (southward propagating waves). Adapted from Spudich and Cranswick (1984a,b).
Figure 5. Location of the SMART-I array on Taiwan and the two main shocks studied. (Inset) the SMART-I array layout. Array consists of three concentric rings of accelerographs near town of Lotung. Adapted from Goldstein and Archuleta (1991).
Figure 6. Vertical accelerograms (upper five traces) and north-south accelerograms (lower five traces) recorded on a subset of the SMART-I sensors for the 1/29/81 Taiwan earthquake (see Figure 5). Duration of a time window used for slowness analysis is indicated by horizontal bracket beneath the traces. From Goldstein and Archuleta (1991).
Figure 7. Slowness distributions in 9 successive time windows of the P waves from the 1/29/81 Taiwan earthquake. At the top of each window T is the center time of the window, dbab is the height of the peak in dB above a background, and se and sn denote the east and north components of slowness of the peak. Maximum slowness of 0.141 s/km in third window corresponds to an apparent velocity of 7.10 km/s; minimum slowness of 0.119 s/km in seventh window implies 8.38 km/s apparent velocity. This shows that there are not resolvable multiple sources radiating at any time on the fault. Adapted from Goldstein and Archuleta (1991).
Figure 8. Map showing location of 1992 Landers epicenter (solid star), aftershocks (dots), Quaternary faults (solid, dashed and dotted lines). Landers surface rupture occurred on the indicated segments of the Johnson Valley fault (JVF), Homestead Valley fault (HVF), Emerson Fault (EF), and Camp Rock fault (CRF). Solid circle is epicenter of the large aftershock that ruptured the Eureka Peak fault (EPF). Inset shows three-stations of the Morongo Valley array (small stars) with 1 km scale bar. From Hough (1994).
Figure 9. Azimuth (measured clockwise from north) toward source of waves (top) and apparent velocity of waves (bottom) observed on the Morongo Valley array in sequential windows of the Yucca Valley aftershock displacement seismograms of the Landers mainshock. Note that azimuth and apparent velocity change around 9 s. Signals before 9 s are interpreted to be caused by direct S waves from the source. Increase of azimuth from 5 to 9 s is consistent with southward propagation of the earthquake rupture along the Eureka Peak fault (Figure 8). Signals after 9 s are caused by surface waves generated at the northwest edge of the Morongo valley. Solid lines drawn by eye through data. From Hough (1994).
Figure 10. Azimuth (measured clockwise from north) toward source of waves (top) and apparent velocity of waves (middle) observed for sequential windows of the displacement seismograms of the Landers mainshock recorded on the Morongo Valley array. The dashed line indicates the expected azimuth for a rupture propagating smoothly along the mapped surface rupture of the Landers event, and the solid line indicates the average azimuth observed from 39-43s (these waves are interpreted by Hough to come from an early aftershock). The dashed line in the middle panel indicates a rough average of the inferred apparent velocities. The bottom panel shows a filtered displacement record at station MVB corresponding to the azimuth and apparent velocity measurements shown above. Apparent velocities are in the 1.0 - 2.0 km/s range. From Hough (1994).
Figure 11. Map of central and southern California showing location of the UPSAR array and Landers epicenter. Inset shows the layout of the UPSAR accelerometers, with a 1 km scale bar.
Figure 12. Results of inversion of observed displacements of the UPSAR array stations (Figure 11) during the Landers earthquake. (Upper left) Vertical displacement from Landers earthquake observed at station P01. (Upper right) Maximum absolute value of any component of the inferred uniform ground strain during the ground motions. (Lower left) Absolute value of the best fitting rigid body rotation of the array about station P01. (Lower right) Misfit ratio, showing proportion of total array station displacements not explained by a uniform rotation and strain. From Spudich and others (1994).
Figure 13. Comparison of stresses determined from inversion of UPSAR surface displacements (observed; solid lines) with stresses inferred from the ground velocity records using equation (4) in text (dashed lines). r is the radial direction and t the tangential direction in a cylindrical coordinate system centered on the epicenter. Tau-rt is the stress caused by Love surface waves, and tau-rr and tau-rt are the stresses caused by Rayleigh waves. Good agreement between the solid and dashed lines indicates the validity of the approximate relationship (4). Tau-rr and tau-rt shifted vertically for clarity. From Spudich and others (1994).
SOME RECENT SITE-SPECIFIC GROUND MOTION EVALUATIONS
SOUTHERN CALIFORNIA EXAMPLES AND SELECTED ISSUES

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ABSTRACT:

This paper examines selected current issues in estimating ground motions in Southern California from the perspective of practicing engineers and scientists. Key issues addressed include: logic tree used in addressing uncertainties in probabilistic seismic hazard analysis, differences between historical versus model-based seismicities, characteristic earthquake recurrence model, near-field directivity effects, spatial variation, and basin effects.

INTRODUCTION

Site specific ground motion evaluations in practice involve probabilistic, deterministic, or a combination of probabilistic and deterministic approaches. The key ground motion parameters include peak values (e.g. peak acceleration and spectral ordinates) and acceleration time histories. These parameters are estimated for a given probability of exceedance at the site in a probabilistic approach and for a postulated earthquake magnitude and distance from the site in a deterministic approach. This paper examines selected ground motion issues using example results from recent site-specific ground motion evaluations in Southern California.

A probabilistic approach in ground motion evaluation usually involves performing probabilistic seismic hazard analysis (PSHA), which also contains most of the elements used in the deterministic ground motion evaluation. The probabilistic approach using PSHA will be discussed first.

PROBABILISTIC SEISMIC HAZARD ANALYSES (PSHA)

The PSHA involves a mathematical process to obtain the mean number of events per year in which the level of a ground parameter Z (e.g., peak acceleration and spectral ordinates) at the site exceeds a specified value z. This mean number of events per year, also referred as "annual

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frequency of exceedance", is designated as "\( \nu(Z \geq z) \)". The inverse of this number is called the "average return period" (ARP) and is expressed in years. Once \( \nu(Z \geq z) \) is obtained, the probability of the level of the ground motion parameter being exceeded over a specified time period, \( t \), can be readily calculated by the following equation based on the Poisson model:

\[
Pr(Z \geq z) = 1 - \exp[-\nu(Z \geq z) \cdot t]
\]

(1)

The elements involved in PSHA are as follows:

1. Defining the location, geometry, and characteristics of earthquake sources relative to the site;
2. Estimating the recurrence of earthquakes of various magnitudes, up to the maximum magnitude, on each source;
3. Selecting attenuation relationships relating the variation of the earthquake ground motion parameter with distance and magnitude; and
4. Performing an analysis which combines various probabilities to obtain annual frequency of the selected ground motion parameter being exceeded at the site.

In the step 4 above, a PSHA at a site due to a particular source involves combining the following three probability functions (Cornell, 1968; Kulkarni et al., 1979):

1. The probability that an earthquake of a particular magnitude will occur on a given source during a 1-year time interval. This probability is expressed in terms of the mean number of earthquakes per year, with a given magnitude on the given source.
2. The probability that the rupture surface is at a specified distance from the site. This probability is assessed by considering both the geometry of the fault and a relationship between the rupture area and magnitude.
3. The probability that the ground motions from an earthquake of certain magnitude occurring at a certain distance will exceed a specified level at the site. This probability is based of the selected attenuation equation considering the site conditions.

By combining these three probability functions for each source, the annual frequency of exceeding a specified level of ground motion at the site is computed. If there are \( N \) sources, then the above process is repeated \( N \) times, covering all the sources, and the contributions are added to obtain the total seismic hazard at the site. A relationship between ground motion level
and annual frequency of being exceeded is obtained by repeating the computations for several
levels of ground motion. The ground motion level corresponding to a specified probability of
exceedance is then obtained from Equation (1) above.

The PSHA summarized above corresponds to a memoryless or Poisson model of PSHA, which
is the most common model of PSHA being used in practice. While other models of PSHA are
sometimes used in practice, they will not be addressed herein. Some of the issues involved in
Poisson versus non-Poisson PSHA are discussed by Cornell and Winterstein (1988).

Logic Tree Approach

In characterizing seismic sources for PSHA, various assumptions and uncertainties regarding the
location, geometry, maximum earthquake magnitude, and recurrence relationships of each fault
often need to be reflected in the analysis. The uncertainties in the various input parameters for
seismic sources are reflected in the PSHA by using a logic tree approach (Kulkarni et al., 1984).
Figure 1 shows an example logic tree for the Sierra Madre fault system used in a recent project.
In the following discussion on various components of seismic source characterization, a logic
tree such as that shown on Figure 1 is used to reflect various assumptions and uncertainties
regarding the seismic source characterization.

Location and Geometry of Seismic Sources

Figure 2 shows key seismic sources in the greater Los Angeles area. The locations and
geometry of seismic sources used in practice are continually updated to reflect the current
thinking of the seismic community. Blind thrusts are of particular interest in many parts of
Southern California. A simultaneous rupture of multiple segments of faults has been allowed
in PSHA through the use of logic tree. The reasonableness of this approach was confirmed by
the Landers earthquake.

Figure 3 shows the three-dimensional model of a segment of the Palos Verdes fault. For seismic
sources that are close to the site, three-dimensional modeling tends to reduce inaccuracy and
inconsistency in the PSHA results due to geometrical effects. For distant faults, piecewise-linear
line modeling is adequate. In either modelling, the effects of various fault segmentations and
seismogenic depths are accommodated in the PSHA through the use of logic trees. Values of
fault segmentation length, seismogenic depth, and dip angle in general specify the geometry of
a given fault.

Maximum Earthquake Magnitude

Although the maximum earthquake magnitude of a fault can be estimated using a variety of
methods, the use of the moment magnitude versus fault area relationships (Wyss, 1979;
Somerville and Abrahamson, 1991; Wells and Coppersmith, 1992) shown on Figure 4 are the
most reliable. Thus, given the geometry of a fault, Figure 4 can be used to estimate the
maximum magnitude associated with the fault. It is noted that on Figure 4 the results of four recent Southern California earthquakes (Joshua Tree, North Palm Springs, Big Bear, and Landers earthquakes) appear to be consistent with the relationships shown on this figure.

Recurrence Relationships

A recurrence relationship represents the cumulative distribution of earthquakes with various magnitudes for each source. For a fault with a given geometry and a given maximum moment magnitude, specifying a recurrence relationship requires a slip rate and a form of the recurrence relationship.

Slip rates are usually estimated for a given fault based on geologic considerations. However, an overall check on the reasonableness of slip rates for a given region can be made if, for example, GPS data are available. For the Los Angeles area, the GPS data between the Palos Verdes Peninsula and the San Gabriel Mountains are used to check the overall values of slip rates assigned to various faults between those two points.

Two recurrence relationships, whose density functions are schematically shown on Figure 5, are often used in the PSHA:

1. Modified or Truncated Gutenberg-Richter relationship (Gutenberg and Richter, 1964): this relationship expressed in terms of three parameters ($a$, $b$, and maximum magnitude) is also referred to as "exponential recurrence";

2. "Characteristic recurrence" relationship (Schwartz and Coppersmith, 1984): this relationship, originally developed by combining data from smaller earthquake magnitudes and geological data from paleoseismic studies, involves five parameters ($a$ and $b$ for the exponential part, and location (defined by the mean characteristic earthquake magnitude), width (assumed to be 1/2 magnitude on Figure 5), and height of the characteristic earthquake part).

In recent PSHA the characteristic recurrence relationship has been given a much greater weight than the truncated exponential one as a consequence of comparison with the historical seismicity as described below.

Although the density functions shown on Figure 5 are for an individual fault, it is difficult to delineate the seismicity associated with an individual fault in a region like the greater Los Angeles area where the tectonic environment is very complicated. However, it is possible to select an area such as the enclosed area shown on Figure 6 and compare the historical seismicity with the seismicity implied by each of the two recurrence models presented in Figure 5. It is assumed in Figure 5 that the maximum magnitude or the mean characteristic earthquake magnitude is independently estimated from relationships such as those shown in Figure 4.
Figure 7 compares the truncated exponential, characteristic, and historical recurrence relationships for the enclosed area shown on Figure 6. The uncertainty bands shown with the historical recurrence relationship are based on Weichert (1980). As can be seen on Figure 7, the characteristic recurrence relationship and the historical recurrence relationship are reasonably compatible. However, the truncated exponential recurrence relationship is significantly different from (and higher than) the historical recurrence relationship.

Figure 8 shows a comparison of the hazard curves from the truncated exponential recurrence relationship and the characteristic recurrence relationship. The higher hazard curve associated with the truncated exponential recurrence relationship is not compatible with the historical seismicity.

It is noted on Figure 7 that the characteristic recurrence relationship implies \( b \) values in the magnitude range higher than about 5 that are significantly lower than that associated with the historical data. Also, the difference between the historical recurrence relationship and the exponential recurrence relationship can be bridged by assuming that some part of the slip is non-seismogenic. However, at this time, in the absence of supporting data, it is prudent to assume that all the slip is seismogenic and use the characteristic recurrence model.

Attenuation Relationships

Attenuation relationships present evolving choices; every year new or revised attenuation relationships become available. Therefore, in general, a number of attenuation relationships should be used in PSHA reflecting the general site conditions and other parameters such as the style of faulting (e.g., higher values for thrust faults). Figure 9 shows the various hazard curves computed in a recent evaluation resulting from the attenuation relationships indicated. In PSHA, various weights are given to different attenuation relationships depending on the particular conditions associated with the study site versus the general knowledge of the ground motion data that went into the attenuation relationships used.

Magnitude-Distance Contributions to Hazard Curves

As part of the PSHA, it is often useful to provide the explicit contributions from various magnitude and distance ranges to a particular point on a hazard curve. Such results can be used to select a deterministic representation of the PSHA results. For example, if a particular point on the hazard curve corresponding to an average return period of 475 years had magnitude-distance contributions shown on Figure 10, then a deterministic representation in this case may be magnitude 7 and distance of less than 15 km.
DETERMINISTIC SEISMIC HAZARD ANALYSES

Deterministic evaluations of ground motion parameters share many components of probabilistic evaluations. In fact, probabilities can be assigned to most of the “events” (e.g., peak horizontal ground acceleration using a median attenuation relationship with magnitude 7 at distance 10 km) associated with deterministic ground motion parameters. However, in the following paragraphs some specific issues in practice more applicable to deterministic seismic hazard analyses are addressed.

Near-Field Effects

In the greater Los Angeles basin and also in the San Diego area, many sites happen to be very close to a significant fault. In such cases it is important to reflect the directivity effects on estimated ground motion parameters particularly if the nearby fault is a vertical strike-slip fault. Figure 11 schematically shows the theoretical reasons for the directivity effects on fault-normal and fault-parallel ground motions (Somerville and Graves, 1993). Figure 12 shows an example of empirical evidence for these directivity effects in the response spectra of the 1979 Imperial Valley earthquake. When directivity effects are present, the fault normal motions are larger than the fault parallel motions for periods longer than about 2 seconds.

In applying these considerations to evaluation of spectral ordinate values for a site close to a significant fault, a response spectrum for the site obtained in a standard manner such as the solid spectrum shown on Figure 13 is often assigned as the fault-parallel motion. For the fault-normal motion the fault-parallel spectrum is adjusted for the directivity effects as shown by the dashed curve on Figure 13. When time histories are required, synthetically generated time histories exhibiting near field effects such as those shown on Figure 14 have been used in practice when appropriate recorded ground motions are not available.

Often synthetic or recorded time histories are adjusted to match “target” response spectrum in a reasonable way using a time domain spectral matching approach (Kaul, 1978, Lilhanand and Tseng, 1987 and 1988)

Spatial Variation

When spatial variation of design or analysis time histories are required, for example, because of widely separated bridge piers, a suite of time histories reflecting incoherency are developed using a procedure by Abrahamson (1992). Figure 15 shows an example of such time histories. Spatial incoherency can be also used to estimate the reduction in high frequency response spectra for large foundations.

Basin Effects

Figure 16 shows the potential importance of basin effects on ground motions based on the study
of the Marina District basin in San Francisco (Graves, 1993) using one-dimensional and three-
dimensional modelling. The 3D model produces the observed duration, while the 1D model
does not. One dimensional site response analyses using SHAKE, for example, are routinely
performed in practice to address site response particularly of soft sites. While three-dimensional
modelling of a basin for basin effects has not been applied to actual projects, project studies have
been made to compare empirically based response spectra with spectra from recorded time
histories considered to reflect basin effects. Figure 17 shows an example of such a comparison
for a site in downtown Los Angeles in which the spectrum computed from an attenuation relation
is compared with the recording of the 1933 Long Beach earthquake. When warranted, some
adjustment to the empirically based response spectra can be made on the basis of the type of
comparison shown on Figure 17.

CONCLUSIONS

This paper examines selected current issues in estimating ground motions in Southern California
from the perspective of practicing engineers and scientists. Key issues addressed include: logic
tree used in addressing uncertainties in probabilistic seismic hazard analysis, differences between
historical versus model-based seismicities, characteristic earthquake recurrence model, near-field
directivity effects, spatial variation, and basin effects.

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Figure 1. Fault tree for seismic source characterization.
Figure 2. Fault map of the greater Los Angeles area.
Figure 3. Three dimensional representation of the Palos Verdes Fault.
Figure 4. Rupture area vs moment magnitude (Mw) relationships.
Figure 5. Density functions for recurrence models.
Figure 7. Comparison of recorded earthquake recurrence in the Los Angeles region with the seismicity calculated from fault slip rates using the characteristic and exponential recurrence models.
Figure 8. Comparison of computed hazard curves using characteristic and truncated exponential recurrence models.
Figure 9. Computed hazard curves for various soil attenuation relationships.
Figure 10. Contribution of various magnitudes and distances to the total hazard.
Figure 11. Schematic diagram of the directivity effects for a vertical strike-slip fault. Left: coincidence of the radiation pattern maximum for tangential motion (SH) and the rupture propagation direction toward the site produce a large displacement pulse normal to the fault. Right: the minimum in the radiation pattern motion (SV) produces small dynamic displacements superimposed on a larger static displacement parallel to the fault.
Figure 12. Average fault normal and fault parallel displacement response spectra at 5% damping for stations within 8 km of the Imperial Fault during the 1979 earthquake. Left: stations without directivity. Right: stations with directivity.
Figure 13. Spectra at 5% damping including near field effect.
Figure 14. Synthetically generated ground motions including near field effect. Motion generated for event with Mw=7-1/2, located 10 km from the site.
Figure 15. Example of suite of incoherent time histories. The motion at Pier 1 is the reference motion.
Figure 16. Marina District basin. Top: structural model. Bottom: Comparison of recorded data (left panel) with synthetic seismograms calculated for a 3D basin model (middle panel) and 1D models (right panel).
Figure 17. Basin effect for the path from Long Beach to downtown Los Angeles.
Utilization of New Developments in Ground Motion Estimation in Engineering Design Practice: Examples for Development of Site-Specific Ground Motions

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Abstract

Examples of development of site-specific ground motions are presented in this paper. The examples are for two case history studies for sites in the western U.S. and the eastern U.S. The case histories illustrate the utilization of recently developed information and techniques in characterizing ground motions for engineering design applications. Aspects of site-specific ground motion estimation that are discussed include earthquake source characterization, ground motion attenuation, probabilistic ground motion analyses, near-source effects on ground motions, site effects on ground motions, and spatial variations of ground motions for extended structures.

Introduction

As earlier speakers in this seminar have described, there have been many significant advances in recent years in our ability to estimate earthquake ground motions. These advances have resulted from both increased understanding of the seismic environment of different regions of the U.S. and elsewhere and development of new techniques and relationships for ground motion estimation. Concurrent with the development of improved analytical tools, many recent earthquakes (Loma Prieta and many others) have provided a greatly expanded number of strong motion recordings that have provided data needed to validate new analytical techniques and provide a firmer empirical basis for ground motion estimation.

This paper presents selected results from two case histories where recently developed information and methodologies were utilized to estimate site-specific ground motions for engineering design applications. These examples are for sites located in the western U.S. and the eastern U.S.

Ground Motion Estimation for Sites in Western U.S.

Comprehensive probabilistic and deterministic ground motion assessments were conducted for seven major bridge crossings of San Francisco Bay. Aspects of these studies, which are described in more detail in Geomatrix Consultants (1993), Power and others (1993, 1994), and Youngs and others (1992), are summarized in the following paragraphs.
Characterization of Earthquake Sources:

Using both geologic and seismicity data, an earthquake source model was developed for the San Francisco Bay region for use in probabilistic and deterministic ground motion analyses (Youngs and others, 1992). Both fault-specific sources and source zones were characterized. The fault-specific sources (Figure 1) were defined by known mapped faults. The spatial distribution of recorded seismicity in the San Francisco Bay Region indicates that the majority of earthquakes occur in close association with the mapped major faults. However, a significant number of events occur in areas where no significant young faulting has been identified. Areal source zones were used to account for the possibility that earthquakes may be generated in the regions between identified faults (Figure 2).

One of the interesting results of the seismic source characterization was the degree of agreement between earthquake recurrence evaluated using geologic slip rate data for faults and the historical seismicity. For example, for the Hayward fault, comparison of recurrence using geologic slip rates with the historical seismicity in a narrow band along the fault is shown in Figure 3. Recurrence curves evaluated using both the more traditional exponential (constant b-value) recurrence model and the recently developed "characteristic earthquake" recurrence model (Schwartz and Coppersmith, 1984; Youngs and Coppersmith, 1985) are shown. The characteristic model places relatively more of the seismic energy release into characteristic earthquakes that are close in size to the maximum earthquake magnitude capable for the fault. Figure 3 illustrates a substantially better degree of agreement between historical seismicity and recurrence modeled using the characteristic earthquake model than the exponential model. The characteristic model was therefore used in modeling recurrence along the faults.

Earthquake recurrence rates of areal source zones were estimated in two ways. Several lines of evidence, including structural geologic data, geodetic data, and plate tectonic considerations, suggest that a component of regional deformation in the Bay Area occurs as fault-normal convergence. The major faults are known from geologic information as well as focal mechanisms to be essentially strike slip. Therefore, the convergence across the region probably occurs within the blocks between strike-slip faults. Estimates of the rate of horizontal shortening across the zone were used to infer fault slip rates, assuming hypothetical reverse faults dipping at 45°. As an equally weighted alternative, earthquake recurrence rates were estimated directly from the observed seismicity within each zone with the regional b-value used to stabilize the recurrence estimates in zones with a limited number of events.

Figure 4 compares the modeled regional recurrence (obtained by summing the recurrence for the individual faults and areal source zones) with the observed regional seismicity rate. There is excellent agreement between the observed seismicity rate and that predicted by the recurrence model.

Uncertainty in the various components of the seismic source model (recurrence rates, maximum earthquake magnitudes, fault segmentation, etc.) was incorporated into the probabilistic analysis using the logic tree technique (Kulkarni and others, 1984; Coppersmith and Youngs, 1986). The logic tree structure is illustrated in Figure 5. The logic tree
approach thus allows multiple credible interpretations for models and parameter values to be incorporated into the analysis.

**Characterization of Ground Motion Attenuation:**

The primary set of attenuation relationships for peak ground acceleration and response spectral values were those developed by Sadigh and others (1993). These relationships were developed based on recorded ground motion data on rock including data from the 1989 Loma Prieta earthquake. In addition, because of the importance of long-period motions to the response of the bridges, simplified numerical ground motion simulation methods were used as an independent check of the long-period attenuation relationships. Numerical simulations based in theoretical Green's functions have been successfully used to model long period (T > 2 seconds) ground motion using simple source and velocity models (e.g., Spudich and Archuleta (1987). The specific method used in this study is that described by Archuleta (1984). This method computes the Green's function for a 1-D crustal model. Figure 6 provides a comparison (in this case for a magnitude 8 earthquake at a distance of 10 km) of long-period response spectral values, obtained by the numerical simulations and those obtained from the attenuation relationships of Sadigh and others (1993). These and other similar comparisons indicated that the empirically-based estimates were adequate at long periods.

**Probabilistic Estimation of Ground Motion:**

Figure 7 illustrates typical results of probabilistic ground (rock) motion analyses at the site of the west (suspension) spans of the San Francisco Bay Bridge. Shown are mean seismic hazard curves for peak ground acceleration and spectral accelerations at periods of 0.30 second and 3.0 second. The total hazard (upper curve) and contributions of various sources to the hazard are illustrated. The dominant contributors to the hazard at the Bay Bridge are the San Andreas and Hayward faults. In general, for all the bridge sites, the source zones contribute little to the hazard in comparison to the faults. From results such as illustrated in Figure 7, response spectra having selected frequencies of exceedance (or selected return periods) were constructed. These were compared with deterministically-estimated response spectra for maximum credible earthquakes and design response spectra were selected on the basis of these comparisons. These results and the selection of criteria spectra are presented in Geomatrix Consultants (1993) and Power and others (1993, 1994).

**Assessment of Near-Source Ground Motions:**

As discussed by Boatwright (this seminar), ground motions may exhibit enhanced long-period spectral content in the near-source region. An assessment was made of these effects in selecting design ground motions for the bridges.

The empirical attenuation relationships used in this study represent ground motions of a randomly oriented horizontal component. However, seismological theory suggests that the strongest shaking for long-period motions will exhibit a preferred orientation in the near source regions of large earthquakes, with this direction controlled by the fault-rupture-site geometry, the rupture direction, and the faulting mechanism. Although the actual direction of rupture of future events cannot be pre-determined, the strike of the major faults and the general mechanism of earthquakes that would occur on them is relatively well determined. A
Numerical modeling study of near field motions conducted for the study indicated that the fault-normal direction is the largest component of long-period motion in the near source region (< 10 km from the rupture) of large magnitude earthquakes. Accordingly, the empirical ground motion data from four well-recorded California earthquakes (the 1979 Coyote Lake, the 1979 Imperial Valley, the 1984 Morgan Hill, and the 1989 Loma Prieta earthquakes) were examined. The results of this examination for data from the 1979 Imperial Valley earthquake at distances to the fault rupture less than 10 km are illustrated in Figure 8. Shown therein is the ratio of the response spectral values of the fault-normal component of ground motion to the average of the fault-parallel and fault-normal components. It can be seen that there is a tendency for the longer-period fault-normal ground motions to be enhanced.

Numerical ground motion simulations were conducted to provide additional insight on the magnitude of the near-source effect. These were similar to those described earlier for assessing attenuation of long-period ground motions. The method described by Bouchon (1981) and modified by Herrmann was used to compute the Green's function for a 1-D velocity model that includes both near-field and far-field terms and surface waves as well as body waves. Ground motions were computed at varying distances and positions from a rupture on a strike-slip fault. Results for a period of 2 seconds are shown in Figure 9 in terms of the ratio of the spectral response of the fault normal component to the average of the fault-normal and fault-parallel components. As shown, for earthquake magnitudes of 6.5 and higher and distances less than 10 km, the fault-normal ground motions are enhanced.

From the empirical and numerical ground motion simulation analyses, it was recommended that the fault-normal component of motion be enhanced for design purposes by approximately 20 percent in the period range of greater than 2 seconds for site-to-fault distances less than 10 km. In application, the actual enhancement for the longitudinal and transverse directions of a bridge were also dependent on the orientation of the bridge relative to the fault.

Assessment of Spatial Variation of Ground Motions:

To obtain input rock motions at the various foundations of the bridges, evaluations were made of wave passage and coherency effects, as discussed by Spudich in this seminar. Apparent wave velocities were estimated on the basis of empirical (array) data and theoretical considerations. A coherency function was selected on the basis of array data (e.g., Abrahamson and others, 1987; Schneider and others, 1992) and is illustrated in Figure 10 for horizontal motions.

Examples of multiple-support time histories of rock motion developed for the San Francisco Bay Bridge are shown in Figures 11, 12, and 13 for horizontal (longitudinal) acceleration, velocity, and displacement, respectively. Station locations are summarized in Table 1. Visually, it is apparent from Figures 11 through 13 that the longer-period velocity and displacement time histories show a progressively greater degree of coherence than the shorter-period acceleration time history. The resulting relative displacement time histories between foundation locations are shown in Figure 14.
Assessment of Site Response Effects on Ground Motion:

An example of the evaluation of local site response effects is provided herein for the San Mateo-Hayward bridge. The subsurface conditions are illustrated in Figure 15. Soft to medium stiff clays (Bay Mud) are present at the Bay bottom and extend to depths up to a maximum of approximately 50 feet below the Bay bottom (beneath a portion of the bridge from piers 15 to 28, the Bay Mud soils have been largely removed by dredging). Beneath the Bay Mud, the soils consist of alternating layers of cohesionless soils (sands, sandy silts) and cohesive soils (clays, clayey silts) to the bottom of borings drilled for the bridge (the maximum depth of the existing borings is approximately 300 feet below the Bay bottom). Based on available data, the unconsolidated sedimentary deposits at the bridge site are estimated to extend to between 600 to 700 feet below sea level. Soil boring data and pile driving records indicate that sandstone bedrock is present at a depth of about 55 feet below the mudline at Pier 1. Thus, the west end of the bridge is underlain by a shallow sloping rock boundary that could lead to potentially significant two-dimensional basin response effects.

One dimensional site response analyses were performed at 11 selected pier locations. Input motions for these analyses were rock motions developed at corresponding pier locations incorporating wave passage and incoherence effects. A two-dimensional response analysis was then conducted to evaluate whether 2-D effects would significantly modify the mudline motions in comparison to those obtained from the 1-D analyses.

Nonlinear 1-D analyses were performed primarily using a modified version of DESRA (Lee and Finn, 1985), designated MARDES (Chang and others, 1990). The MARDES code incorporates the Martin-Davidenkov model (Martin, 1975) instead of the hyperbolic stress-strain model. The Martin-Davidenkov model allows a description of secant shear modulus as a function of shear strain, similar to the description required in SHAKE. A two-dimensional response analysis was performed using the computer code FLUSH. The two-dimensional model of the subsurface soil deposits at the west end of the bridges used in the analysis is shown in Figure 15. Soil motions at selected pier locations computed from the two-dimensional analysis were compared with those from the one-dimensional analyses. The two-dimensional effects were quantified by computing ratios of Fourier spectra of the motions from the two-dimensional analysis to those from the one-dimensional analyses. The analysis results indicated that the two-dimensional effects were small in this case expect at Piers 1 and 4 (near the edge of the sloping rock boundary).

Time histories of acceleration at mudline (transverse components) for selected pier locations (Piers 1, 9, 13 and 19) developed from the analyses are shown on Figure 16. Response spectra (5% damped) of these motions are shown in Figure 17. For comparison, the time history and response spectrum of the rock motion at Pier 1 are also shown on Figures 16 and 17. The results illustrate that site and basin response effects result in substantial variations of mudline motions from location to location (i.e. substantial additional incoherence in comparison to rock motions) along the alignment of the bridge due to the highly variable subsurface conditions. The presence of soft soil deposits results in amplification of long-period motions and deamplification of high-frequency motions. In addition, the duration of strong shaking is increased on the soils.
Probabilistic and deterministic ground motion analyses were conducted to develop design response spectra for a site in southeastern Arkansas near the Mississippi River. The site location is shown in Figure 18. Only the probabilistic analyses are discussed herein.

**Characterization of Earthquake Sources:**

As discussed by Johnston and Nava (1990) and Adams and Basham (1994) (papers in this volume), discrete seismic sources (active faults) generally have not been identified in the central and eastern United States. As a result, sources are usually characterized as relatively large zones considered to have a relatively uniform earthquake potential within them (in terms of frequency of occurrence and maximum size of earthquakes) on the basis of information on tectonics, geology, and historical seismicity.

The seismic sources zones characterized for the region surrounding this site are shown in Figure 18. These source zones represent a synthesis of the result presented in two comprehensive evaluations of seismic hazard in the central and eastern United States conducted by the Electric Power Research Institute (EPRI, 1987) and the Lawrence Livermore National Laboratory (LLNL), (Bernreuter and others, 1989), as well as subsequent generalized evaluations (Johnston and Nava, 1990; and Mitchell and others, 1991). These previous studies presented various interpretations of seismic source zones for eastern North America. The source zones outlined on Figure 18 are considered to be the best defined source zones on the basis of both historical seismicity and tectonic features. Some of the detailed source zones defined in the EPRI and LLNL studies were combined into single zones to provide a more conservative analysis of seismic hazard at this site.

As shown in Figure 18, the site is located in the Ouachita fold belt seismic zone, a region of low seismicity. North of and close to the site is the low-seismicity Southern Reelfoot rift seismic zone, while farther north (180 km from the site) is the relatively high seismicity New Madrid seismic zone, which was the source of the 1811-1812 New Madrid earthquake sequence.

On the basis of analysis of historical seismicity, earthquake recurrence relationships were developed for the various seismic sources zones. The historical seismicity for the New Madrid seismic zone is shown in Figure 19 along with a fitted recurrence curve having an exponential magnitude distribution. Also shown in Figure 19 is the inferred recurrence of large magnitude earthquakes from paleoseismic data. Two recurrence relationships were used for the New Madrid seismic zone. One consisted of the exponential model fitted to the seismicity and extrapolated to the maximum magnitude ($m_s$ approximately 7.5) for this source zone; the other consisted of a characteristic magnitude model which fits the moderate-magnitude seismicity data yet also fits the paleoseismic data for the occurrence of large earthquakes. The two recurrence relationships used are shown by the upper and lower curves in Figure 20. An exponential magnitude distribution fitting historical seismicity was used to model recurrences for the other seismic source zones.
The modeling of uncertainty in the seismic source characterization is illustrated in the logic tree in Figure 21. For all sources, two methods, equally weighted, were used to analyze completeness in the seismicity catalog. Figure 21 elaborates on the modeling of the Ouachita fold belt seismic source zone. As shown, it was modeled as both a single zone (the entire source) and two zones roughly separated by the Mississippi River, since the region to the east appears to have lower seismicity than the region to the west (Figure 18). These alternative models were given equal weight. To account for uncertainty in maximum magnitude, two values were used, weighted as shown. The wide range in modeled recurrence rates and b-values reflects the limited amount of seismicity data.

Characterization of Ground Motion Attenuation:

The site is characterized as a deep soil site. Representative shear wave velocity profiles for Mississippi Embayment sediments and for the underlying crust are shown in Figure 22. Ground motion attenuation relationships for the site were characterized on the basis of (a) published attenuation relationships for deep soil site ground motion in the eastern U.S. and (b) numerical modeling of ground motions. The published attenuation relationship was that developed by Boore and Joyner (1991).

Numerical modeling of ground motions was carried out using the approach developed by Silva and Lee (1987). Their approach combines the stochastic estimation of rock motions outlined by Boore (1983) and nonlinear site-specific wave propagation through the use of an equivalent-linear formulation for one-dimensional wave propagation in a layered medium (Silva, 1989; Silva and others, 1989). The resulting model provides a means of obtaining site-specific estimates of ground motion. The analyses performed for this study were conducted using an extended version of the Silva and Lee (1987) model that include the crustal wave propagation modeling techniques of Ou and Herrmann (1990) to account for direct and super critically reflected waves within the crust. These effects have been shown to be significant in estimating ground motions at distances of 80-200 km from the source (Burger and others, 1987; Somerville and others, 1990). Numerical modeling was conducted over a range of magnitudes and distances and the results were compared to the relationships of Boore and Joyner (1991). From those results, a second, modified attenuation relationship was developed. Both sets of relationships are shown in Figure 23. As indicated in the logic tree in Figure 21, the two relationships, equally weighted, were used in the probabilistic analyses.

Probabilistic Analysis Results:

Hazard curves obtained from the analysis for peak ground acceleration and response spectral accelerations for three periods of vibration are shown in Figure 24. The bands around the mean curves reflect the uncertainty in input parameters as incorporated in the logic tree. Source contributions to the mean hazard curve are shown in Figure 25. The hazard is dominated by the New Madrid seismic zone because of its much higher rate of earthquake occurrence than the other seismic zones.

From the hazard curve results for a number of periods of vibration, equal hazard response spectra were constructed. These response spectra are shown in Figure 26 for return periods varying from 100 to 5000 years. Superimposed on the 500-year return period
spectrum is the Uniform Building Code spectrum shape for deep firm soils (type S2) anchored to the 500-year peak ground acceleration. A reasonable degree of agreement can be seen in this case; except for a somewhat higher site-specific shape in the long period range, reflecting the dominance of large magnitude, distant earthquakes in the New Madrid seismic zone. It should also be noted that the spectrum shape is highly dependent on the seismic environment in the region of a site. For sites where the seismic hazard is dominated by small to moderate magnitude earthquakes (such as many eastern U.S. sites), the long period spectral content can be well below that of the standard UBC spectrum shapes.

Acknowledgments

Many colleagues of the author at Geomatrix Consultants participated in the studies summarized herein, including C.-Y. Chang, Shyh-Jeng Chiou, Kevin Coppersmith, Shyh-Jeng Chiou, Laurel DiSilvestro, Dario Rosidi, Ross Sadigh, Charles Taylor, Zhi-Liang Wang, and Robert Youngs. The firm of International Civil Engineering Consultants participated in studies for San Francisco Bay Bridges, especially characterization of spatial variations of ground motions and development of multiple support acceleration time histories. Consultant Norman Abrahamson participated in numerical ground motion modeling and spatial variation of ground motion studies for San Francisco Bay Bridges. Consultant Walter Silva conducted numerical ground motion modeling for the Arkansas site. Studies for the San Francisco sites and the Arkansas site were sponsored by the California Department of Transportation (Caltrans) and the U.S. Army Corps of Engineers, respectively.
References


Geomatrix Consultants, 1993, "Seismic Ground Motion Studies for Major Northern California Bridges," Reports to California Department of Transportation for San Francisco
References (continued)

Bay Bridge crossings, prepared in association with International Civil Engineering Consultants.


References (continued)


### TABLE 1

Pier Locations For Spatial Variations Of Ground Motion
At West San Francisco Bay Bridge

<table>
<thead>
<tr>
<th>Ground Station No.</th>
<th>Pier Designation</th>
<th>Alignment Station No.</th>
<th>Distance From S.F. Anchorage Along Bridge Alignment (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S.F. Anchorage (West end of bridge)</td>
<td>49+66.38</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>A-B</td>
<td>53+92.19</td>
<td>425.81</td>
</tr>
<tr>
<td>3</td>
<td>W1</td>
<td>58+29</td>
<td>862.62</td>
</tr>
<tr>
<td>4</td>
<td>W2</td>
<td>70+00</td>
<td>2,033.62</td>
</tr>
<tr>
<td>5</td>
<td>W3</td>
<td>93+10</td>
<td>4,343.62</td>
</tr>
<tr>
<td>6</td>
<td>W4</td>
<td>105+55</td>
<td>5,588.62</td>
</tr>
<tr>
<td>7</td>
<td>W5</td>
<td>118+00</td>
<td>6,833.62</td>
</tr>
<tr>
<td>8</td>
<td>W6</td>
<td>141+10</td>
<td>9,143.62</td>
</tr>
<tr>
<td>9</td>
<td>Y.B. Anchorage (East end of bridge)</td>
<td>152+70</td>
<td>10,303.62</td>
</tr>
</tbody>
</table>
Figure 1 Regional Source Fault Map in San Francisco Bay Area
Figure 2  Map of the San Francisco Bay Area showing independent earthquakes, fault corridors, and areal source zones. Fault corridors define the area within which seismicity is assumed to be related to fault-specific seismic sources.
Figure 3  Comparison of recurrence rates developed from independent seismicity and from fault slip rates for the Hayward fault. Predicted recurrence rates are shown for the characteristic earthquake and exponential magnitude distribution models.
Figure 4 Comprehensive recurrence model for the Central San Francisco Bay Area.
Figure 5  Generic logic tree used in Bay Bridges study to characterize seismic sources for probabilistic seismic hazard analysis.
Figure 6  Comparison of the numerically simulated long period motions with the empirical median spectrum for M 8 at R = 10 km.
Figure 7
Contributions of various sources to mean hazard at the west end of the West San Francisco Bay Bridge. Shown are results for peak acceleration and 5 percent-damped spectral accelerations at periods of 0.3 and 3.0 seconds.
Figure 8 Near Source Ground Motion Data for Imperial Valley, 1979 Earthquake; Distance to Fault Less Than 10 KM
Figure 9  Results of Near-Source Ground Motion Simulation, Period = 2.0 Seconds
Figure 10  Plane-Wave Coherency Functions for Horizontal Motions
Figure 11  Generated Coherency-Compatible Acceleration Time-Histories at All 9 Stations (Longitudinal Component)
Figure 12 Integrated Velocity Time-Histories of the Generated Coherency-Compatible Motions at All 9 Stations (Longitudinal Component)
Figure 13 - Integrated Displacement Time-Histories of the Generated Coherency-Compatible Motions at All 9 Stations (Longitudinal Component)
<table>
<thead>
<tr>
<th>STATION 1 (S.F. ANCHORAGE)</th>
<th>REL. DMAX = 0 cm</th>
<th>WEST BAY BRIDGE - LONGITUDINAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>STATION 2 (PIER A-8)</td>
<td>REL. DMAX = 5.4 cm</td>
<td></td>
</tr>
<tr>
<td>STATION 3 (PIER W1)</td>
<td>REL. DMAX = 7.6 cm</td>
<td></td>
</tr>
<tr>
<td>STATION 4 (PIER W2)</td>
<td>REL. DMAX = 18.4 cm</td>
<td></td>
</tr>
<tr>
<td>STATION 5 (PIER W3)</td>
<td>REL. DMAX = 32.6 cm</td>
<td></td>
</tr>
<tr>
<td>STATION 6 (PIER W4)</td>
<td>REL. DMAX = 36.2 cm</td>
<td></td>
</tr>
<tr>
<td>STATION 7 (PIER W5)</td>
<td>REL. DMAX = 42.7 cm</td>
<td></td>
</tr>
<tr>
<td>STATION 8 (PIER W6)</td>
<td>REL. DMAX = 53.1 cm</td>
<td></td>
</tr>
<tr>
<td>STATION 9 (Y.B. ANCHORAGE)</td>
<td>REL. DMAX = 54.0 cm</td>
<td></td>
</tr>
</tbody>
</table>

Figure 14  Relative Displacement Time-Histories for Stations 2 through 9 Relative to Station 1 (Longitudinal Component)
Figure 15  Two-dimensional finite element idealization at San Mateo - Hayward bridge site.
Figure 16 Acceleration time histories of mudline motion (transverse component) at pier locations 01, 09, 13, and 19 at San Mateo-Hayward Bridge site.
Figure 17  Response spectra (5% damped) of mudline motion (transverse component) at pier locations 01, 09, 13, and 19 at San Mateo-Hayward Bridge site.
Figure 18  Historical seismicity and seismic source zones for the south-central United States. A - Great Plains, B - Nemaha ridge, C - Ozark uplift, D - St. Louis-Wabash, E - south-central, F - New Madrid seismic zone/Reelfoot rift complex, G - Southern Reelfoot rift, H - New York-Alabama lineament, I - eastern Piedmont/Mesozic basin, J - Ouachita fold belt, K - southern Gulf Coast, L - New Mexico-Texas, M - Oklahoma anadogen.

Legend:
- Pre-1811
- 1811-1812
- Post-1960
- Site
- N/A

Magnitude
- 8
- 7
- 6
- 5
- 4
- 3
- 2
- 1
- 0

Distance:
- 0
- 100
- 200
- 300
- 400
- 500 km
Figure 19  Earthquake recurrence relationships for New Madrid source zone. Box at lower right is inferred rate for large magnitude events from paleoseismic data.
Figure 20  Recurrence relationships used to model New Madrid seismicity.
Figure 21  Logic tree used to model input parameter uncertainty for probabilistic seismic hazard analysis.
Figure 22. Velocity profiles used to compute site-specific ground motions. (a) Generalized shear wave velocity profile for Mississippi Embayment sediments. (b) Crustal velocity profile for Mississippi Embayment.
Figure 23  Attenuation relationships
Figure 24: Computed hazard curves for peak acceleration and 5% damping spectral accelerations at periods of 0.1, 0.5, and 2.0 seconds. Shown are the mean hazard curves and the 5th, 15th, 85th, and 95th percentile hazard curves developed from the distributions of the input parameters.
Figure 25 Contributions of the main seismic sources to the total mean hazard.
Figure 26  Equal-hazard spectra (5% damping) for return periods of 100, 200, 500, 1,000, 2,000, and 5,000 years, and comparison with Uniform Building Code Spectral Shape.
ESTIMATION OF EARTHQUAKE GROUND MOTIONS
IN THE PACIFIC NORTHWEST

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Abstract

The purpose of this paper is to review procedures to estimate ground motion in the Pacific Northwest. This review is confined to the characterization of potential seismic sources and the estimation of strong ground motion. Issues related to these two aspects of seismic hazard analysis are discussed.

Seismic Source Characterization

The tectonic environment of the Pacific Northwest is distinctly different from that of other parts of the contiguous United States. The main tectonic feature in the Northwest is the Cascadia subduction zone (CSZ), which is primarily responsible for the regional seismicity and volcanic activity. During the last 10 years, researchers have uncovered geologic evidence that the CSZ has generated great earthquakes of moment magnitude $M > 8$ (Atwater, 1987a,b; 1992; Darienzo and Peterson, 1987, 1990). For seismic design, it is therefore prudent to consider the possibility of future events of this size. However, while most scientists and engineers studying the seismic hazard in the Pacific Northwest have accepted the possibility that a great CSZ earthquake could occur, there is no consensus regarding the size, location and recurrence of these events. The range of magnitudes estimated for great CSZ events vary between 8 and 9+ (WPPSS, 1988; Heaton and Hartzell, 1986; Weaver and Shedlock, 1991; Rogers, 1988). Although the differences in ground motion between $M = 8$ and $M = 9+$ events may not be large for most of the frequency band engineering interest, the impact these two events would have on the Pacific Northwest are vastly different. The $M = 8$ event would effect only a portion of the Northwest, whereas the $M = 9+$ event would impact the entire region from northern California to southern British Columbia.

The eastern extent of the rupture area associated with great CSZ earthquakes is another important issue that bears directly on the severity of ground motion in, for example, the urban areas along the Interstate 5 corridor and between Vancouver, B.C. and Salem, Oregon. The estimates of the eastern limit of fault rupture vary by 50+ km from locations west of the coast to locations in the middle of the Coast Ranges or Olympic Mountains (Hyndman and Wang, 1993; Heaton and Hartzell, 1986; Weaver and Shedlock, 1991; Cohee et al., 1991; Atwater, 1991 – pers. comm.). This difference is more significant for the estimation of higher frequency ground motions which attenuate more rapidly with distance than for lower frequency motions.

The recurrence of great CSZ earthquakes is the remaining significant issue, which affects the calculation of probabilistic ground motions, i.e. motions with given probabilities of being exceeded during some future time period. The paleoseismic record and other evidence suggests that the average recurrence interval is on the order of 500–600 years; however, the observed intervals in the paleoseismic record have been as short as 100–200 years and as long as 1,000+ years (Atwater, 1987a,b; Darienzo and Peterson, 1987, 1990; Adams, 1987). Because the last event appears to have occurred about 300 years ago, it should not be surprising if another event occurs next year or 700 years from now. A Poisson model of recurrence based on an average recurrence interval of 500 years, or a simple time-dependent model that considers the time since the last event in addition to the average recurrence interval, yields a probability...
of a great CSZ earthquake in the next 50 years which is on the order of 10 percent. This probability level is similar to that associated with the ground motions which modern code-designed buildings should be able to resist without collapse. However, at these levels of ground motion significant damage may occur which could potentially have a severe economic impact.

Further research should be directed toward understanding the recurrence of great CSZ earthquakes in space and time. Since the last event, presumed to have occurred ~300 years ago, movement on the order of 10 m may have accumulated as stored strain energy along the interface between the Juan de Fuca and North American plates (assuming that most of the 3—4 cm/yr plate convergence rate has been stored in this manner rather than being released as aseismic creep). A sudden displacement of 10 m at the interface could be generated by a $M > 8$ earthquake. If such an event were to occur, it would be worthwhile from an emergency planning standpoint to know beforehand the likely extent of the damage that might be expected. Thus, additional building-response and loss-estimation studies are suggested. The results of these studies could then be used to establish or revise short-term and long-term planning and mitigation efforts.

The great CSZ earthquake is not the only seismic event of concern. Large historical earthquakes of $M \geq 6\frac{1}{2}$ have occurred in Puget Sound and the Cascades (Noson et al., 1988), and paleoseismic data suggest that a major earthquake occurred on the Seattle fault about 1,000 years ago (Bucknam et al., 1992). The intraplate portion of the CSZ in Puget Sound generated the 1949 Olympia ($M = 7.1$) and 1965 Seattle ($M = 6.5$) earthquakes and numerous smaller events. In a study of the earthquakes recorded between 1970 and 1990 by the University of Washington seismographic network, Weaver and Shedlock (1991) noted that no event of $M \geq 4$ has occurred on the intraplate CSZ at depths shallower than 45 km. Furthermore, few intraplate CSZ events have been observed south of Puget Sound. This lack of intraplate seismicity south of Puget Sound is a major issue facing engineering seismologists performing probabilistic seismic hazard analysis in, for example, the Portland, Oregon region. The development of a recurrence model for intraplate CSZ earthquakes in this region is highly uncertain. Postulating a recurrence rate lower than that for the intraplate CSZ in Puget Sound might be justified on the basis of the different geometries of the downgoing Juan de Fuca plate in both regions. The bend and warping of this plate in Puget Sound may be responsible for the intraplate seismicity observed in this region; the apparent lack of such contortions in the plate south of Puget Sound may explain the lack of intraplate seismicity in that region. While this explanation appears reasonable, it does not offer much help to the engineering seismologist who must develop a recurrence model for intraplate CSZ earthquakes in the Portland region. Assuming an intraplate CSZ activity rate for Portland that is identical to the rate for Puget Sound is probably too conservative; however, estimating a recurrence rate based on the observed sparse seismicity may be too unconservative. More research on intraplate seismicity in other subduction zones might offer some insights to this problem.

The identification of sources of potential earthquakes within the North American crust in the Pacific Northwest is more difficult than in California, for example, where the location of the active or potentially active faults are fairly well known. The identification of active faults in the Northwest is complicated by the vegetation cover. In addition, the seismogenic depths of earthquakes cover a wide range ($0 — 30$ km) (Noson et al., 1988), and therefore large deep events could occur with little or no surface expression.

Two significant faults have been discovered which pose a challenge to the engineering seismologist estimating probabilistic ground motions in Seattle and Portland. These faults are the Seattle and Portland Hills Anticline faults. Recurrence rates have been estimated for these faults based on displacement (slip) rates estimated from geologic data. When these recurrence rates are substituted into standard seismic hazard analysis models, large ground motions in Seattle and Portland are computed.
However, these slip rates are uncertain; in addition, the percentage of slip that may be aseismic is also unknown. Nonetheless, decisions must be made on whether to model these faults separately or to implicitly include them as one of many sources comprising a tectonic province. If a particular fault is modeled separately, then the recurrence model selected should yield estimates of seismicity for the fault that are not substantially greater than the historical or paleoseismic record of seismicity observed on the fault. What level of seismicity constitutes "substantially greater" is a matter of judgment.

The aforementioned uncertainties in seismic source characterization in the Pacific Northwest raises the question of how this problem should be approached for estimating future ground motions in this region. For critical structures, such as nuclear plants, an approach involving multiple expert opinions has been used (e.g. Geomatrix, 1988). A similar type of approach could be implemented to develop the next generation of ground-motion maps for the building codes. However, such a formalized and relatively time-consuming approach is impractical for most projects in which consultants perform probabilistic seismic hazard analysis to estimate ground motions for structural design or retrofit. For these situations the consultant must necessarily make judgments based on published information and, if possible, discussions with recognized experts.

Ground-Motion Estimation

Measurements of strong ground motion in the Pacific Northwest during large earthquakes are limited to the few accelerograms recorded during the 1949 Olympia and 1965 Seattle earthquakes on the intraplate CSZ. These data indicate that the attenuation of ground motion from CSZ earthquakes is different from the attenuation observed during California earthquakes. On the other hand, the few accelerograms recorded during the 1993 M = 5.6 Scotts Mills, Oregon earthquake, a relatively shallow event in the North American crust, suggest that for these types of earthquakes, the ground-motion attenuation in the Pacific Northwest and California are comparable.

The limited accelerogram data in the Pacific Northwest are obviously insufficient by themselves to develop ground-motion attenuation equations for the region. For shallow crustal earthquakes, engineering seismologists have generally used published attenuation equations derived mostly from California accelerogram data. For interplate and intraplate CSZ earthquakes, attenuation equations derived from accelerogram data recorded during subduction-zone earthquakes around the Pacific Rim have been used to estimate ground-motions (Crouse, 1991a,b; WPPSS, 1988; Youngs et al., 1988). These empirical attenuation models are generally preferred in practice. However, another type of model has also been used. This model, called the Band-Limited-White-Noise model, has been combined with random vibration theory to estimate ground motions (Silva et al., 1989).

To compliment the results of traditional empirical approaches, semi-empirical approaches have also been used to estimate the ground motions from great CSZ interplate earthquakes (Cohee et al., 1991; Heaton and Hartzell, 1986). In the semi-empirical approach, accelerograms are selected from smaller earthquakes and superimposed according to seismological principles to simulate the ground motions from a larger event.

Although the methods described above provide estimates of ground motions in the Pacific Northwest, these estimates are more uncertain than, for example, the ground-motion estimates in California or Japan, where abundant accelerogram databases exist. The import and use of data from other regions to estimate ground motions in the Northwest result in additional uncertainties because these data are never completely representative of the source characteristics and regional and local geologies in the Pacific Northwest. Furthermore, the source characteristics of great CSZ earthquakes will remain largely unknown.
until such an event occurs and is well recorded. Therefore, instrumentation should be placed at strategic locations throughout the Northwest to capture this or other events that may occur.

Another issue is the effect of local geology on strong ground motion. The Duwamish/Green and Willamette/Columbia river valleys of Seattle and Portland, respectively, are essentially basins with softer sediments overlying predominantly dense glacial tills. Analytical calculations have been performed to evaluate the potential for ground-motion amplification, but ground-motion measurements are needed for verification. The surficial geology of most of the Seattle urban area is dense glacial till that can extend to depths greater than 200 feet. Geotechnical engineers generally regard this material as bedrock even though its shear-wave velocities \((V_s)\) are generally in the range of 1200–2200 fps. Strictly speaking, a deep deposit of till \((\text{depth}>200 \text{ ft})\) with \(V_s < 2500 \text{ fps}\) would be classified as Soil Type S\(_2\) according to the soil classification in the 1991 Uniform Building Code (UBC). This classification, as opposed to a bedrock Soil Type S\(_1\) classification, results in a 20% increase in the UBC base shear coefficient, which some argue is too conservative. Reluctance to classify tills as S\(_2\) sites is understandable because the amplification factors associated with typical S\(_2\) sites were derived from accelerogram data recorded mostly in California where the S\(_2\) sites are generally softer alluvial sands and clays which extend to great depths or which overlie weak sandstone or siltstone deposits. Such deposits usually exhibit greater increases in shear-wave velocity with depth than do glacial tills, and therefore would tend to amplify the ground motions to a greater extent. However, until more shear-wave velocity measurements are made at till sites and until more ground motions are measured, the amplification effects of till will remain uncertain.

Conclusions

Significant advances have been achieved in seismic source characterization and ground-motion estimation in the Pacific Northwest during the last 10 years. The new information on the CSZ and ground-motion attenuation are gradually being incorporated into engineering practice for the purpose of computing site-specific ground motions for seismic design and retrofit of individual structures or for use in the development of regional ground-motion maps for inclusion in the seismic codes. However, many issues are unresolved, and therefore additional research and deployment of instrumentation to record strong ground motions are recommended.

References


IMPLICATIONS FOR THE SEISMIC DESIGN OF BUILDINGS

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Abstract

The USGS is complimented for this collection of papers related to the recent developments in earthquake ground motion estimation. Much of the information presented in this seminar series will eventually lead to new design techniques and code provisions. The work related to new developments in estimating site response effects on ground motions by Borcherdt and the description of the USGS ground motion mapping efforts by Frankel were found to be particularly useful at the present time. These efforts were reviewed in detail and discussed herein. Observations were made on their application and impact to design standards in general and various suggestions offered for their improvement.

Introduction

Modern building codes related to seismic design are written to guide the construction of new buildings. They are written using a consensus process by researchers and practicing engineers and are intended for use by design professionals. In general, they represent the available collective experience of the profession on the behavior of structures in past earthquakes, presented in a manner that is applicable to all forms of construction. This code development process is ongoing and carried out by various professional associations and agencies. Seismic design codes are continually undergoing revisions and improvements as new information becomes available. Much of the information presented in this seminar series will eventually lead to new design techniques and code provisions.

Over the past few years, there has been a growing interest in developing more sophisticated design procedures and codes that consider a variety of performance standards for buildings subjected to earthquakes. Traditionally, codes focused their attention protecting life with only minor consideration given to minimizing property damage. As earthquakes have occurred and the significance of damage in terms of repair cost, loss of investment and loss of use are realized, new procedures have emerged that strive to achieve a full range of performance objectives. Recently, the National Institute of Standards and Technology developed standards for the seismic safety of federal buildings in which they defined a vocabulary for the range of performance objectives currently in use. The categories include fully functional, immediate occupancy, damage control, and substantial life-safety. The relationship between these performance objectives and the available standards is shown in Table 1.

Major earthquakes affecting areas with modern construction continue to show that most buildings do not experience catastrophic damage. If fact, the areas most affected are usually small in comparison to the regions that feel the shaking. Observations of these events has shown that successful performance of buildings during earthquakes depends on the intensity and characteristics of the ground motion, basic strength of the main structural elements, the strength and ductility of the connections, the building configuration, the material type and quality of construction, the interconnection of the structural elements, and the extent to which the nonstructural elements and contents of the building are adequately anchored and braced. Each of these issues is addressed to various levels of completeness in the various available
standards. Obviously, the higher the performance objective, the more sophisticated and complete the attention to these key details must be.

The traditional techniques used for design purposes appear to do an excellent job of achieving their lifesafety goals. In fact, more often than not, buildings built to modern codes appear to perform better than expected when subjected to strong shaking. It is also common for older buildings that do not meet the modern codes to perform quite well. With the current focus on performance objectives comes the need for a better understanding on why some buildings are damaged in earthquakes and others are not.

It is becoming very apparent that one of the key reasons that buildings often perform better than expected is related to the intensity and characteristics of the shaking experienced during a particular earthquake. In the early 1900s, speculation after the 1906 earthquake led to the conclusion that the ground shaking was equal to accelerations on the order of 10% g, based on the performance and strength of buildings that withstood the earthquake successfully. At the time, it was thought that all earthquakes produced similar accelerations and it was suggested that most areas of the nation could experience similar events, and the technical debates related to the effects of soft ground, flexible versus stiff structural systems, appropriate levels of base shear, and the extent of the earthquake hazard fell into full swing (Dewell, 1925).

With the advent of earthquake recording instruments, our understanding of the potential for and intensity of earthquakes has been dramatically refined. Over the years, much of this information has been used by the design professionals to improve their design standards and codes. The current generation of codes, first introduced in the mid-1970s as ATC-3, made good use of the then available information and has remained the basis of the current design provisions.

Recent large earthquakes, including Loma Prieta and others, have produced an extraordinary and growing set of strong motion records that are rapidly expanding our understanding of the characteristics of earthquakes and the extent of their influence. Recent work by the USGS and others has led to substantial new developments in earthquake ground motion estimation that will be particularly useful to building design professionals. This seminar series includes an excellent summary of much of the work that is underway. The work reported on related to new developments in estimating site response effects on ground motions (Borcherdt) and ground motion mapping (Frankel, et al.) appear to be of immediate significance to building design professionals.

Site Amplification Due to Local Geologic Conditions

Evidence from virtually all earthquakes shows that most damage and life loss is concentrated in areas underlain by soft soil. This observation has led to a variety of design implications that range from treating all sites as equally poor (Anderson, 1952) to suggestions that land use planning standards not permit buildings to be built on soft sites. The current seismic design standard, as presented in the 1991 Edition: NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings (NEHRP91) classifies sites in one of four classifications with modification factors that range from 1 to 2.0.

The paper by Roger Borcherdt reports on the coordinated efforts of many investigators, researchers and practitioners that has led to some very credible recommendations on how to better account for the amplification effects of local geologic deposits. Because these recommendations are consistent with current standards, consensus-based, and have come from a combination of analytical studies and strong motion records taken in the Loma Prieta earthquake, they represent a new generation of information that is worthy of consideration and inclusion in the current design standards.
All too often, the current site descriptions that are used in the code to characterize local geologic conditions are misinterpreted to the detriment of the project budget. It is not uncommon for the design professional and the building official to select soil amplification factors that are unnecessarily high to be conservative when controversy arises over the actual classification. Even when local site shear wave velocities are measured and available, controversy remains. The new site classification system proposed by Borcherdt and others is a significant improvement that will go a long way toward eliminating the misclassification of the local geology in an affordable manner. Since it stands as a consensus-based development, it is expected to be readily acceptable to design professionals and building officials. An application has already been accepted for the next revision of the SEAOC Bluebook.

Included in the Borcherdt paper is a proposed update of the NEHRP recommended provisions - Section 4.2.1. The most significant modifications of the proposal are the introduction of two site amplification factors, $F_v$ and $F_a$. $F_a$, as an amplification factor applied to the maximum value of $C_s$, represents a new variation in the design coefficient basis that will have substantial impact on the design base shear for buildings in low seismic zones. $F_v$ is similar to the current NEHRP91 $S$ factor though it is recommended to have a broader range. Both $F_a$ and $F_v$ are proposed to vary as a function of both input ground motion and site classification. The proposed classifications are, in general, equivalent for firm sites (values of $S_1$ and $A$), lower for hard rock sites ($A_o$), and higher for soft sites with lower input accelerations. This apparent calibration to the design values for buildings in zones with highest base shears is consistent with the fundamental design assumptions of the code (ATC, 1978).

To demonstrate the implication of this proposal, recommended base shear design values were calculated for four actual buildings that are representative of the range of design values changes. They include a concrete tilt-up warehouse, a two-story steel moment frame office with wood diaphragms, a six-story concrete frame and shear wall office, and a twenty-story steel moment frame office. Design base shear values were calculated for each building, for each of the soil types and seven of the NEHRP91 seismic zones. The results are listed in Tables 2, 3, 4, and 5.

The concrete tilt-up building is a typical warehouse-style construction with a tall story height and plywood roof diaphragm. This type of construction is governed by the maximum base shear value and, therefore, affected by the newly defined $F_a$ value. As shown in Table 2, the base shears in high seismic zones ($A_a=.4$) are equal or slightly less, and in low seismic zones as much as 200% greater. The implication of these changes, however, is likely minimal since the cost of this style of construction is generally not sensitive to the base shear value. The higher loads may lead to some increases in the diaphragm nailing schedules, as well as the strength of the diaphragm ties and chords. These new factors also imply that damage to this style of construction may be somewhat more than previously expected on soft sites as compared to firm sites.

The two-story, steel moment frame office building with wood diaphragms is also a style of construction that is governed by the maximum required base shear. In this case, because the building design may be governed by the code drift limitations, the impact of the proposed changes in the lateral loads, as shown in Table 3, will have a greater impact on the structural system. However, because this building type is lightweight, the effect of these changes may be somewhat tempered if the design is governed by wind. These new factors also imply that damage to this type of construction will be more than previously expected on soft sites.

The six-story concrete building has a concrete moment frame in the longitudinal direction and a set of concrete shear walls in the transverse direction. The base shear calculations shown in Table 4 are for the shear wall direction. The changes in base shear are related to both NEHRP91 base shear formulas and both site amplification factors. It is questionable whether the increase in base shear for $A_a=V_a=.4$ and...
soil type B is necessary. It is expected that these changes will result in significant increases in structural costs in all zones due to the increased base shears. These increases will be caused by the need for more shear resisting elements and more extensive foundations. Damage estimates for this type of construction may be somewhat more than previously expected though this style of construction is not particularly sensitive to higher peak ground accelerations.

The twenty-story steel moment frame building will be subject to increases in the design base shear for softer sites in all seismic zones. The proposed changes will likely result in significant increases in structural costs in the mid-to-high seismic zones. These costs will be associated with heavier steel columns and beams that will be required to meet the drift limits under the higher forces and perhaps increased foundation costs. For areas with low seismicity, the wind design criteria often govern. While these proposed changes seem to indicate that damage will be more than previously expected for all softer sites, the historical performance of this type of building does not necessarily support that trend.

The general result of using these adjusted soil factors for design will be to narrow the spread of design values used across the various zones of seismicity. Similarly, if used as intended, they will increase the estimates of damage for buildings on softer sites that experience lower input ground motions. Given the current trends toward performance-based criteria, and the interest in keeping various types of facilities functional for lower level earthquakes, engineers would do well to pay attention to the fact that much higher amplifications are expected for lower levels of earthquakes. Admittedly, the overall impact of these conclusions is somewhat overshadowed by the minor premium that seismic design requirements bring to a new project in low zones of seismicity. However, each of these suggestions will have considerable impact when used in the evaluation strengthening of existing buildings.

The next step needed in the process of estimating site effects is to quantify the effects of changes in frequency content that occur for various local geologic conditions. The historic performance of buildings seems to indicate that certain classes of structures are more vulnerable on softer sites than others. Previous attempts to relate building period and site period to the design base shears proved unworkable. Intuitively, however, the observed variations in frequency content and duration of strong motion records and their expected impact on structural systems cannot be overlooked. Perhaps the new site classification system and the growing suite of strong motion records will soon yield a plausible technique for including this observation effectively in the design of new buildings and the evaluation and strengthening of existing buildings.

Ground Motion Mapping

When earthquakes occur, the resulting damage patterns strongly suggest that the ground motion attenuate rapidly and the area affected by the strongest shaking is quite limited. This fact, coupled with the growing body of knowledge about earthquake sources and their probability of generating significant events should lead to a set of design value maps that reflect fairly small zones with the highest expected intensity located adjacent to recognized sources with substantial reduction in the expected intensity beyond.

The two principal approaches to mapping ground-motion hazards discussed in the paper by Frankel and others clearly lead to this type of refined information. They report that the probabilistic technique is preferable because it not only considers the ground motion from various scenario earthquakes, but also considers their relative likelihood. They report that USGS has been using the probabilistic-based techniques since 1976 to prepare their "scientifically derived maps of probabilistic ground motions." Their expectation, of course, was that the design professionals would take this information and create the design values maps that are needed for the seismic design standards and codes. Unfortunately, this has not been the case.
While USGS researchers have been carefully constructing maps using probabilistic methods to quantify the seismic hazard on a uniform national basis, the design engineers continue to cry out that the resulting values are too high. They routinely retreat to their own seismic hazard mapping committees and draw their own versions of maps that continue to appear in the seismic design standards and codes. These maps tend to simplify the hazard maps, create far fewer zones, adopt minimum levels of ground motion in some areas of low seismicity and truncate all ground motions higher than 0.4 g, regardless where they occur. In recent years, this stalemate has reached unmanageable proportions and has begun to erode away the very credibility of the code development process.

Art Frankel and his colleagues have presented in this seminar series a very thorough and well-documented statement on the past, present, and future of ground motion mapping at the USGS. It appears that they have cleared up many of the questions that have existed for years related to the basis for the values that have been proposed. The probabilistic approach that is presented, and apparently will continue to be the basis of their efforts, is certainly accepted and consistent with most of the site-specific ground motion estimates that are produced for special facilities today. It appears that the fundamental discrepancy between what the engineers expect to use and what the USGS techniques produce is buried in the assumptions and procedures, not tied up in the "relevant issues" of seismic hazard assessment.

It would be interesting to focus on the various assumptions used to model the seismic hazard in various parts of the United States and take issue with those that are controversial while defending those that are agreeable. Fortunately (or unfortunately), such a discussion is beyond the expertise of this author. While that discussion is important and must continue, it does not address the key problem that currently exists in translating the "scientifically correct probabilistic result" to usable design values maps. To be successful, the translation needs to be a joint effort that respects the needs of both efforts while focusing on a common result. It cannot be done after the maps are drawn, the contours of incredible pseudo accelerations are set and there seems to be no rational way to affect the needed reduction.

Frankel states that a maximum magnitude earthquake must be selected for each source zone to complete the probabilistic methodology modeling. Design engineers continue to argue that the probabilistic techniques are yielding ground motion estimates that far exceed the potential of the maximum earthquakes. In an effort to clarify the seismicity assumption in the eyes of the design engineers, the assumed maximum earthquakes should be clearly stated and characterized as a part of the mapping process so that the design standards and codes can develop procedures for covering the worse possible earthquakes. Engineers understand all too well that buildings must be designed, detailed and constructed such that given the largest expected earthquake, no life-safety concerns develop. A building's occupants must be safe inside the building during such an event, and able to exit when it is over; even if the building is a total financial loss. It is quite likely that prescriptive requirements will be developed to protect against collapse in these maximum events, and these requirements should be based on the expected events, not the processed maximum accelerations such as those with a 10% exceedence in 250 years.

Because earthquake continue to occur on "newly discovered" faults, there is some valid concern on the part of the design professionals that the probabilistic method will overlook significant unknown hazards. Their response has often been to add background seismicity to their probabilistic models to represent the unknown fault sources. It is not clear if the USGS efforts are doing this. It would be better to agree on a reasonable level of background seismicity for each region of the nation than to have the local design professionals add minimum design values to increase the design values to their level of expectation.

The probabilistic methodology also relies on a probability density function to describe the distribution of ground motion for an earthquake with a specific magnitude and distance. The variability appears to be characterized by a log-normal distribution of ground motions about the mean value. Frankel shows an
idealized example of this function in figure 5 of his paper. This log-normal function, centered about the mean value determined from averaging the logarithms of ground motion assigns a portion of the probability density to very high ground motions based on the shape of the distribution, regardless of their possible occurrence. It appears that the extraordinary accelerations calculated as a result of the analysis, and reported on the NEHRP91 Maps 10 and 11, are a function of the shape of the log-normal curve. While this assumption is likely quite reasonable for ground motions with maximum acceleration values that can be delivered to the site, there should be some limit to the maximum value considered based on the local geologic structure and the frequency band of accelerations that influence structural behavior. This limit should be built into the analysis, perhaps with a different probability density function, or with a limit in the return period considered for the maps. In this way, the limits on effective acceleration that are needed to calibrate the impact of ground motion to the historic behavior of buildings will at least in part occur within the ground motion estimation routines. It remains the task for the engineer to deal with the translation of the predicted free-field ground motion to the effective motion that the structure experiences.

Frankel reports that they are presently incorporating geologic information in their hazard maps for California. This yields a probabilistic ground motion prediction that includes the amplification due to local site conditions. The resulting maps are a fascinating example of the future of seismic zonation. Their degree of local accuracy could lead to much better land use planning decisions, better decisions related to siting critical facilities, and much better use of the limited resources available for the seismic strengthening of deficient facilities. Unfortunately, the pseudo acceleration response values reported appear to be way too high when compared to other ground motion estimates. It appears that the site amplification factors used did not vary with the level input acceleration as has been recommended by Borcherdt and others. This assumption along with the assumption related to the log-normal probability distribution function needs to be revisited in light of the historic performance of buildings in these regions.

Conclusions

The USGS is to be complimented for the this collection of papers related to the recent developments in earthquake ground motion estimation. Much of the information presented in this seminar series will eventually lead to new design techniques and code provisions. The work related to new developments in estimating site response effects on ground motions by Borcherdt and the description of the USGS ground motion mapping efforts by Frankel were found to be particularly useful at the present time. These were reviewed in detail and the following general observations and conclusions offered.

Conclusions related to estimating site response:

1. The historical behavior of buildings in earthquakes convincingly demonstrates that the local geologic conditions are a dominate factor in understanding performance.

2. Current site factors in the NEHRP91 do not go far enough to quantify the observed effects.

3. Recent analytical studies and strong motion records have led to a new set of site factors that better describe the expected variation.

4. Through a consensus process, these factors have been adapted to the NEHRP91 in an appropriate manner that is generally consistent with the basic design assumptions contained therein.
5. The general results of using these adjusted soil factors for design will be to narrow the spread of design values used across the various seismic zones. Their use should also improve estimates of damage that are made related to various levels of ground shaking.

6. The next round of improvements in this area should deal directly with the impact of modifications in the frequency content of the strong motion as it is transmitted through the local site geology.

Conclusions related to ground motion mapping:

1. The seismic zone maps used for design purposes are out of step with the technology available to represent zones of significant strong motion. In many cases, the current maps are too conservative. In others, they do not adequately represent the hazard.

2. While probabilistic techniques for developing design ground motions are generally accepted as the preferred procedure for developing national maps, their application in the past without input from design professionals has led to unacceptable results.

3. The maximum earthquakes assumed during the map development for each seismic source should be published along with the maps. Engineers should develop standards for using these values to assure protection against collapse.

4. The use of log-normal probability distribution functions needs to be reviewed and limits added to the potential for high accelerations. Similarly, there is a need to include some level of background seismicity in areas where the resulting strong motion estimates appears to be too low.

5. The proposed new generation of seismic hazard maps that include site amplification effects needs to consider and include the work by Borcherdt and others.

References


<table>
<thead>
<tr>
<th>Performance Objective</th>
<th>Previously Defined Standards</th>
<th>Specific Concern</th>
<th>Example Occupancies</th>
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<td>Fully Functional</td>
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<td>Resist a minor earthquake without damage Resist a moderate earthquake without structural damage but with some nonstructural damage Resist a major earthquake with damage but without collapse</td>
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**TABLE 1: PERFORMANCE OBJECTIVE ADVISORY MATRIX**
**Taken from NIST (1993)**
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Concrete Tilt-up Warehouse

<table>
<thead>
<tr>
<th>Facts</th>
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<th>91 NEHRP Base Shears</th>
<th>Borcherdt Base Shears</th>
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<td>0.10 0.11 0.13 0.16 0.18</td>
<td>0.90 1.00 1.20 1.40 1.60</td>
</tr>
</tbody>
</table>

Implications

1. Small increase in structural cost in low seismic zones and soft sites
   This type of construction is not sensitive to lateral forces
   May require some increase in diaphragm nailing, as well as, tie and chord strength
2. No significant change in mid to high zones
3. Minor variations in damage for soft sites in low zones should be expected

**TABLE 2**: BASE SHEAR COMPARISON FOR A CONCRETE TILT-UP WAREHOUSE
### Two-story, Steel Moment Frame Building with Wood Diaphragms

<table>
<thead>
<tr>
<th>Facts</th>
<th>Zone Factors</th>
<th>91 NEHRP Base Shears</th>
<th>Borcherdt Base Shears</th>
<th>Ratio of Change</th>
</tr>
</thead>
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<tr>
<td></td>
<td>Aa</td>
<td>Av</td>
<td>$S_1$</td>
<td>$S_2$</td>
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<td>T = 0.67</td>
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<td>R Factor</td>
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<td></td>
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<td>0.3</td>
<td>0.06</td>
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</tbody>
</table>

**Implications**
1. Moderate increase in structural cost in low zones since building is drift sensitive
2. Larger structural section likely required for stiffness
3. Some minor savings in highest zone with very firm or soft sites
4. Significant variations in damage as a function of site conditions should be expected

**TABLE 3: BASE SHEAR COMPARISON FOR A TWO-STORY, STEEL MOMENT FRAME BUILDING WITH WOOD DIAPHRAGMS**
### Six-story, Concrete Frame and Shear Wall Building

<table>
<thead>
<tr>
<th>Facts</th>
<th>Zone Factors</th>
<th>91 NEHRP Base Shears (%)</th>
<th>Borcherdt Base Shears (%)</th>
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<tr>
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<td>R Factor 5.5 (shear walls)</td>
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<td></td>
<td>0.3 0.3</td>
<td>0.10 0.12 0.14 0.14</td>
<td>0.08 0.10 0.14 0.15 0.16</td>
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<td>0.08 0.09 0.11 0.13 0.15</td>
<td>0.85 1.00 1.20 1.40 1.60</td>
</tr>
</tbody>
</table>

**Implications**

1. Significant increase in structural cost in zones with increased base shear. More and stronger shear walls likely required. Foundations will be much more significant.

2. No savings expected in high zones.

3. Minor variations in damage expected, this style of construction is not particularly sensitive to variations in effective peak accelerations.

**Table 4: Base Shear Comparison for a Six-Story, Concrete Frame and Shear Wall Building**
### Zone Factors 91 NEHRP Borcherdt et al

<table>
<thead>
<tr>
<th>Site Classes</th>
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<th>Borcherdt et al</th>
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### 20-story, Steel Moment Frame Building

<table>
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<td>0.3</td>
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</table>

#### Note
Wind will likely govern design in low seismic zones

#### Implications
1. Significant increases in structural costs in the mid to high zones
   Increase forces will lead to larger steel sections due to drift limits
   Some minor increase in foundations possible
2. No significant increase expected in low zones because seismic does not govern
3. Variations in damage may be more limited than implied

**TABLE 5: BASE SHEAR COMPARISONS FOR A 20-STORY, STEEL MOMENT FRAME BUILDING**
SEISMIC PERFORMANCE EVALUATION OF LONG-SPAN BRIDGES

Roy A. Imbsen and Wen David Liu
Imbsen & Associates, Inc.
Sacramento, California

ABSTRACT

Following the 1989 Loma Prieta earthquake, it was evident that important transportation structures must be designed not only to prevent collapse, but also to maintain its function following a major earthquake. This later design requirement on functionality poses a greater demand on the analytical capability and the modeling of important details. The damage on the East Bay Bridge is a prime example. Recently, several long span bridges have been evaluated for seismic retrofit design including: Golden Gate Bridge; Mississippi River Bridge, Memphis; Benicia-Martinez Bridge; and Vincent Thomas Bridge, Los Angeles. The seismic evaluation involves: development of spatially-varying ground motion input at multiple bridge supports; analyses of soil-structure interaction effects at each foundation; development of 3D analytical model of the structure-soil system; identification of structural seismic vulnerability and the sensitivity to relative displacement input at the multiple supports. Spatially-varying multiple-support excitations were specified as the input using coherency models derived from dense array data in Taiwan and California. Using the Benicia-Martinez Bridge as an example, this paper describes the systematic methodology for the seismic evaluation of long extended bridge structures subjected to multiple support seismic excitation.

INTRODUCTION

Following the Loma Prieta earthquake of October 17, 1989, the Governor's Board of Inquiry into the causes of structure failures recommended the following seismic safety criteria:

"that all transportation structures be seismically safe and the important transportation structures maintain their function after an earthquake."

To achieve the seismic performance goals in both the safety limit state and the function limit state, the seismic evaluation methodology must be refined to better represent the 3D structural behavior and the realistic seismic input. For long structures and long-span structures, the effects of the spatially-varying ground motion input at the multiple supports are very important to the seismic performance. Three effects can be identified:

1. Quasi-static response due to relative displacement between supports;
2. Dynamic response due to in-phase, i.e., average, ground acceleration input;
3. Dynamic response due to out-of-phase ground acceleration input.
Detailed seismic evaluations have been conducted recently for several long span bridges including:

- Golden Gate Bridge (1125 ft. – 4200 ft. – 1125 ft.), San Francisco.
- Mississippi River Bridge on I-40 (900 ft. – 900 ft., Tied Arch), Memphis.
- Benicia-Martinez Bridge, I-680 (330 ft. – 429 ft. – 7 @ 528 ft. – 429 ft.), California.
- Vincent Thomas Bridge (507 ft. – 1500 ft. – 507 ft.), Los Angeles.

The Benicia-Martinez Bridge was the first of the eight toll bridges in California for which detailed seismic evaluations were carried out to identify seismic vulnerabilities, to develop retrofit schemes and to estimate construction cost. This paper describes the seismic evaluation procedure used in the seismic performance assessment.

Seismic Performance Criteria

The Seismic Performance Criteria established by Caltrans for design and evaluation of bridges is shown in Table 1. Seismic performance is assessed at the following two levels of earthquake ground motion:

1. Function Evaluation Earthquake having 40% probability of occurring during the useful life of the bridge; and
2. Safety Evaluation Earthquake which is either the maximum credible earthquake based on the conventional deterministic assessment or an earthquake with an average return period of 1000 to 2000 years.

The seismic performance requirement for important bridges was significantly raised to the following level:

1. The structure should remain essentially elastic under the Functional Evaluation Earthquake; and
2. The structure should provide service to normal traffic almost immediately following the Safety Evaluation Earthquake. Any damage incurred should be repairable with limited loss of service, i.e., short closure time.

Objectives

Benicia-Martinez Bridge is considered a critical link for the regional economic activities and is therefore classified as an important bridge. A detailed site-specific seismic hazard assessment had been completed. (Geomatrix, 1992). The following two tasks were carried out:

1. Assessment of the seismic vulnerability of the structure in its current configuration under both the Functional and Safety Evaluation earthquakes; and
2. Development of a global seismic retrofit concept and feasible structural strengthening details that meet the above seismic performance goal.
Location and Description of the Bridge

Benicia-Martinez Bridge (George Miller, Jr. Bridge) is a high-level, deck-type, welded truss bridge with welded girder approach spans. This 6,215-foot-long structure carries I-680 over the easterly end of Carquinez Strait between the cities of Benicia in Solano County and Martinez in Contra Costa County. The general plan of the bridge is shown in Figure 1. The soil profile at the bridge is shown in Figure 2. The truss section of the bridge, spanning a stream width of about 5,000 feet, consists of seven 528-foot spans, two 429-foot spans and one 330-foot span. All trusses are 33 feet in depth (center-to-center of chords). The 330 foot span is simply supported on fixed bearings on Pier 3 and expansion bearings on Pier 4. Four separated continuous spans are supported on fixed bearings over Piers 5 and 6, Piers 7 and 8, Piers 9 and 10, Piers 11 and 12, respectively. In between these continuous spans, "drop-in" truss spans are suspended from the cantilever ends of continuous spans or piers by fixed and expansion hinges. Two trusses at a spacing of 42 feet center to center are braced with top and bottom lateral bracings, end sway frames at piers, and intermediate sway frames at a spacing of 66 feet. The truss spans are supported by reinforced concrete cellular pier shafts on cellular reinforced concrete footings resting on groups of eight or ten 6-foot diameter, concrete-filled steel caissons.

Contract work for the initial 4-lane, 67-foot wide structure began in 1959 and was completed in 1962. Earthquake restrainers and reinforced concrete support and shear blocks were added in 1980. In 1991, the structure was widened equally on each side to its present width of 77'-6" to accommodate six lanes of traffic. Figure 1 denotes the original construction and the 1991 widening. The substructures of the main truss spans are briefly described below:

Piers 3 and 13. These piers are stepped at the top to support girder spans toward the approaches and the much deeper truss spans toward the center of channel. Reinforced concrete cellular construction was utilized in the four cell major shafts (15 foot by 57 foot) and in the narrow upper shafts (6 foot by 57 foot). A reinforced concrete spread footing on tremie seal supports the pier. The widening contract added reinforced concrete "ears" to the pier shafts to support the new exterior girders in the approach spans, but made no changes in the foundation.

Piers 4-12 (Water Piers). Like Piers 3 and 13, the base section of the water piers is cellular, consisting of four in-line reinforced concrete cells. The top 40 feet, however, consists of two split cellular columns (each 15 foot by 15 foot) spaced 42 foot center to center. The footings are also of reinforced concrete cellular construction, each with 18 cells. The footings are 25 feet deep with the top 10 feet designed to extend above the mean sea level. Six-foot diameter steel caissons (eight at Piers 4 and 12, ten at the other piers) were sunk to bedrock with a combination jetting and driving action after holes were drilled. Sixty inch diameter "sockets" were drilled into the bedrock to anchor the caissons. The lower 20 feet of caisson and socket were reinforced as a spiral R.C. column. Caissons were filled with concrete to a depth of eight feet into the footing. Concrete was placed in the cells (where caissons were located) to a total depth of 15 feet from the bottom of footing. A 12-inch cap was placed over the entire footing. The typical caisson is shown in Figure 3.

All bearings under the original contract were of steel. The truss bearings are of the rocker type, measuring 3'-8" from top of rocker to base of masonry plate. Wide flange floorbeams (45 inch web) at 33-foot centers span between trusses, while 27WF94 stringers at 6-foot centers frame into the floorbeams and carry the deck load. Truss hinges are located in Spans 4, 6, 8, 10
and 12. Trusses were fabricated of T1 steel ($f_y = 90,000$ psi) for most of the main members. Lesser stressed members were fabricated from A242 steel. Chords and diagonals utilize an H-type cross section through most of the frames. The truss connections are bolted with high strength bolts. Under the widening contract, floorbeams were extended by bolting new wide flange sections to the existing beams. An additional stringer was added at each side to carry the widened deck. Upper and lower chord members were strengthened in the vicinity of supports and truss hinges.

In 1980, cable earthquake restrainer units were added to girders at the abutments and land piers. Reinforced concrete support blocks were placed under each girder (1 inch clear). Blocks were doweled into the face of the abutment footing. Cable earthquake restrainer units were also placed at the water piers to restrain the trusses vertically and horizontally. Support blocks of reinforced concrete were placed under the lower chord members of the trusses near the faces of piers. Cover plates were added to chord members in Span 10 in the vicinity of the truss hinges.

SITE-SPECIFIC EARTHQUAKE GROUND MOTIONS

A seismic hazard study at the project site was completed by Geomatrix Consultants (1992). The results were used to define both the Safety Evaluation and the Functional Evaluation earthquake ground motions for this study.

Design Response Spectra at Rock Outcrop

Using the probabilistic approach, all potential seismic sources were systematically taken into account. Equal hazard response spectra (with 5% damping) were developed for average return periods of 100, 300, 500, 1000 and 2000 years. In the deterministic approach, maximum credible earthquake events associated with three dominant faults were established: the San Andreas Fault, the Hayward Fault, and the Green Valley Fault. The 84th percentile spectra for horizontal motions are shown in Figure 4. The ground motion parameters summarized in Table 2 include peak ground acceleration, peak ground velocity, peak ground displacement, and strong motion duration for each of the three events. It is clear that, for all structural periods of interest, the response spectral values corresponding to the Green Valley event exceed the other two earthquake events. It should be recognized that the San Andreas and Hayward events have much longer shaking duration, and these events should be taken into account in the final design phase.

Because of the close proximity of the Green Valley fault to the bridge site (about 3 km), the fault strike direction could have a significant effect on the response spectra of the two horizontal components. To account for this aspect, three target spectra were selected, one for each ground motion component. Typically, the fault-normal component has higher spectral values for periods greater than 2 seconds. Since the difference between the two components was not large for most structural period range, the average spectrum of the two horizontal components was used.
Safety Evaluation and Functional Ground Motions

A comparison of the probabilistic and deterministic estimates of ground motion spectra for the Green Valley event showed that the 84th percentile spectra corresponded to an average return period of 1000 years and was therefore chosen as the basis for the Safety Evaluation ground motion. For the Functional Evaluation earthquake, the useful life of bridge must be defined. Given a 40% probability to occur, the average return periods for several values of useful life are:

<table>
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<tr>
<th>Useful Life (Year)</th>
<th>Average Return Period (Year)</th>
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<tr>
<td>50</td>
<td>100</td>
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<tr>
<td>75</td>
<td>150</td>
</tr>
<tr>
<td>150</td>
<td>300</td>
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</table>

For all toll bridges across the San Francisco Bay, the useful life was considered as 150 years due to their importance. However, in the case of the existing Benicia-Martinez Bridge, a new parallel bridge will be completed in a few years. For this redundant transportation link, it is worthwhile to consider a shorter useful life of the existing bridge, e.g. 50 years. The ground motion return periods are 100 years for 50 year useful life and 300 years for 150 years useful life. The equal-hazard acceleration spectra are shown in Figure 5 in linear scale for return periods of 100, 300 and 1000 years. The ratios of the 1000 year spectrum to that of the 100 year and 300 year events are relatively constant over the entire period range. It was concluded that, for rock motions, the 100 year event spectrum is about 40% of the 1000 year event, and the 300 year event is about 60% of the 1000 year event.

Theoretically, the site response study and soil-structure interaction study should be carried out to determine the support input motions corresponding to these lower levels of rock motion. However, it was found that the nonlinear soil response was not too critical to the support motions. In the subsequent study, the support motion time histories for the Functional Evaluation event are approximated by applying a scale factor to the Safety Evaluation event.

Spatially-Varying Ground Motions

Ground motion time histories at the multiple supports of the bridge were developed in accordance with the following steps:

1. Selection of rock motion time histories from natural strong motion recordings of historical earthquakes with similar characteristics (magnitude, distance to site, etc.);
2. Modification of the selected time histories to be compatible with the target spectra; and
3. Development of incoherent rock motion time histories at all supports which were compatible with the selected coherency function.

These rock motions were used in the subsequent soil-structure interaction analyses to determine the support input motions for the Safety Evaluation event.
METHOD OF ANALYSIS

To perform the seismic evaluation of the bridge structure in an efficient manner, the general substructure method described below was used. The entire structure-foundation-soil system is divided into two subsystems:

1. the truss spans and piers, and
2. the caisson, surrounding soil and underlying rock.

The footings serve as the natural division between the two subsystems. To completely define the bridge structure, the boundary conditions at the footings must be defined. These include foundation stiffness and damping matrices at each pier footing as well as the effective support input motions. These boundary conditions will be derived from the soil-structure interaction analyses of the caisson and soil subsystem.

Soil-Structure Interaction Analyses

The cross section of a typical pier extending from bedrock to the roadway level is shown in Figure 6. The analysis method can be briefly summarized as follows:

Step 1. Conduct the free-field site response analysis at each pier location to determine the soil response motions subjected to the design rock motion input. This is schematically shown in Figure 7. Liquefied soil layers were identified. The motions at top of the firm soil layer were determined. These analyses were performed using the iterative, equivalent linear soil-layer seismic analysis program SHAKE. With an understanding of the nonlinear soil responses at each pier location, a linearized soil profile was determined which is described by the effective shear modulus and damping ratio consistent with the soil strain level.

Step 2. Conduct impedance soil-structure interaction analysis to determine the stiffness and damping matrices for each caisson group foundation. This is shown schematically in Figure 8. To perform these analyses, the 3D soil-structure interaction analysis program SASSI (Lysmer et al, 1981) was used to model the caisson group and soil. The linearized soil profile determined in Step 1 was used which also accounted for the liquefied soil layers. The foundation impedance functions are complex-valued and frequency dependent. To allow for the nonlinear analysis of the overall structural system, frequency-independent values of these impedance coefficients were selected in accordance with the natural frequencies of the most dominant modes of the structure.

Step 3. Conduct kinematic (scattering) soil-structure interaction analysis to determine the effective support input motions at each pier footing. Soil motions obtained in Step 1 were used in this calculation. This is shown in Figure 9.

With information obtained from the above three-step soil-structure interaction analysis, one can then perform the global seismic response analysis of the structure as shown in Figure 10. To evaluate member internal forces of the foundation elements (i.e., caissons), it is necessary to
carry out additional studies of the foundation-soil model. Two sources of excitation may contribute to the internal forces of caissons:

- interaction forces from the superstructure, and
- seismic inertia loads acting on the caissons due to the surrounding free-field soil motions.

Based on studies carried out in this project, it is concluded that, at critical sections along the length of caisson, member internal forces are governed by the interaction forces from the superstructure. In other words, the foundation elements can be evaluated based on forces or displacements of pier footings calculated from the superstructure analysis.

Multiple-Support-Excitation Seismic Response Analysis

For long extended structures subjected to out-of-phase input motions at multiple supports, there are two response aspects induced:

- vibrational effect – common to all seismic responses; and
- quasi-static effect – caused by the differential support movements.

In the conventional dynamic analysis, only the vibrational component of the response (sometimes referred to as the relative displacement responses) is considered. To account for both the vibrational effect and the quasi-static effect, the total displacement formulation was implemented in the IAI-NEABS program (an enhanced version of NEABS program) using the direct-integration time history analysis method (IAI, 1991). The quasi-static response components were evaluated by imposing unit displacement successively at each degree of freedom of each support point.

ANALYTICAL MODEL OF THE BRIDGE

A three-dimensional model of the trusses, bracings and deck system was developed based on as-built drawings which included 2267 truss members and 72 zero-length hinge elements. The total number of degree of freedom in the global model was 4572. A three span section of the model is shown in Figure 11. The pier-foundation substructure was modeled using beam-column elements for the pier, and caisson groups were modeled as foundation springs and dashpots using generalized 6x6 matrices. At the fixed hinges, the trusses are connected through a 10 inch φ pin; at the expansion hinges, the suspended spans are hung by 5 foot link members. The details for the fixed and expansion hinges are shown in Figure 13. At each hinge, fixed or expansion, the transverse degree of freedom was restrained. For these bearings, translational stiffnesses were calculated based on the flexibility of the anchor bolts. Rotational stiffnesses of each bearing were ignored.

For the concrete piers, both gross section properties and cracked section properties were used to assess the sensitivity of the structure response. As it turned out, the structural responses were dominated by the foundation flexibilities and not sensitive to the concrete section properties used.
In the global model, caisson foundations and the underlying soil and rock were represented by a stiffness matrix and a damping matrix at each pier location. In addition, the structural damping was specified in accordance with the conventional Rayleigh damping, i.e.,

\[ C = aM + bK + C_F \]

where \( C_F \) is the equivalent viscous damping derived from the foundations, \( M \) and \( K \) are mass and stiffness matrices of the structure, and \( a \) and \( b \) are numerical values reflecting the amount of structural damping included. \( a \) and \( b \) values selected were 0.2233 and 0.00444 corresponding to modal damping ratios between 3% to 5%.

Dynamic Behavior of the Bridge

The truss structure is composed of one simple span between Piers 3 and 4, and four continuous spans. Three drop-in spans are suspended from the cantilevered continuous spans. The structure is on a straight alignment. The longitudinal vibration modes and the transverse vibration modes were uncoupled. Typically, longitudinal modes are coupled with the vertical vibration of the truss system. The first four longitudinal modes involve the vibration of each of the four continuous spans individually. The periods of these modes are:

<table>
<thead>
<tr>
<th>Location</th>
<th>Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piers 9 - 10</td>
<td>( T_6 = 2.08 )</td>
</tr>
<tr>
<td>Piers 5 - 6</td>
<td>( T_8 = 2.03 )</td>
</tr>
<tr>
<td>Piers 7 - 8</td>
<td>( T_{12} = 1.80 )</td>
</tr>
<tr>
<td>Piers 11 - 12</td>
<td>( T_{17} = 1.52 )</td>
</tr>
<tr>
<td>Pier 4</td>
<td>( T_{16} = 1.62 )</td>
</tr>
</tbody>
</table>

Due to the expansion hinges or bearings, the longitudinal vibrations of Piers 3, 4 and 13 were uncoupled from the rest of the structure with vibrational periods of 1.21, 1.62 and 1.37 seconds, respectively. The lowest longitudinal mode shape is shown in Figure 14. At higher modes, vibration of different parts of the structure may occur simultaneously either in phase or out of phase. The out-of-phase modes are very important for long structures with substantial out-of-phase ground motion input.

Lateral vibrations of the piers and trusses are typically coupled with the torsional responses of the truss system particularly in the suspended span. This is very critical because the suspended spans are supported on the cantilever trusses through the pins and hangers which are non-redundant. The first five modes in the transverse vibration have periods ranging from 2.13 seconds to 1.66 seconds:

<table>
<thead>
<tr>
<th>Location</th>
<th>Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier 6</td>
<td>( T_5 = 2.13 )</td>
</tr>
<tr>
<td>Piers 10, 11</td>
<td>( T_7 = 2.05 )</td>
</tr>
<tr>
<td>Piers 9</td>
<td>( T_{13} = 1.77 )</td>
</tr>
<tr>
<td>Piers 7, 8</td>
<td>( T_{14} = 1.72 )</td>
</tr>
<tr>
<td>Piers 4, 5</td>
<td>( T_{15} = 1.66 )</td>
</tr>
</tbody>
</table>

The shape of the lowest transverse mode is shown in Figure 15.
MULTIPLE-SUPPORT-EXCITATION SEISMIC ANALYSIS

Effective Multiple-Support Input Motions

Detailed soil-structure interaction analyses were carried out at each pier location to establish the effective input motions which were consistent with the specified rock motion spectrum and the appropriate coherency functions. The support input motion parameters (i.e., maximum displacement, velocity and acceleration) are summarized in Table 3.

The relative displacement time histories between any two support points were calculated for both longitudinal and transverse components. The maximum relative displacements between any two pier footings are summarized in Table 4. It should be noted that during the earthquake excitation, soil liquefaction occurred at the upper soil layers under Piers 6, 7, 8 and 9. This resulted in 12 inches relative longitudinal displacement and 9 inches relative transverse displacement between Piers 9 and 10 as shown in Table 4.

Quasi-Static Responses

The quasi-static responses are the structural responses (displacements and member forces) caused by the relative displacement input between supports. If all supports move in unison, the structure moves as a rigid body and there should be no internal force induced in the structure. By imposing unit displacement in each direction at each bridge support successively, the sensitivity of the structural responses to relative support displacement input was established. The key response parameters are the forces developed at all the truss bearings and hinges.

For the support displacement imposed in the longitudinal direction, only longitudinal bearing forces were induced. Due to the expansion hinges, only bearings within the same continuous span were affected. The maximum bearing force was 722 kip per 12 inches relative longitudinal displacement between adjacent piers 528 feet apart. As discussed earlier, a relative displacement of 12 inches did occur between Piers 9 and 10, and would contribute significantly to the total seismic demand (as much as 25%).

For support displacements specified in the transverse direction, bearing forces in all three directions (longitudinal, transverse and vertical) were induced. Maximum bearing forces induced were 373 kip longitudinal at Pier 4, 367 kip longitudinal at Pier 13, and about 110 kip transverse for bearings at Piers 6, 8 and 10. Vertical bearing forces induced by transverse support displacement were typically less than 50 kip per 12 inch support displacement. The maximum longitudinal shear force induced in the fixed hinge was 730 kip.

Maximum quasi-static forces induced in the bearing and hinge connections by 12 inch support relative displacements imposed in the longitudinal and transverse directions, respectively, are summarized in Table 5. The effect of relative vertical displacement between adjacent supports was negligible.
SEISMIC PERFORMANCE UNDER SAFETY EVALUATION EARTHQUAKE

Both quasi-static response and dynamic response are included in the dynamic analysis. All subsequent analysis results were based on the total responses. However, the contribution of the out-of-phase support input can be inferred based on the quasi-static responses reported earlier.

Displacement Responses

The entire time history of the pier responses was calculated. As shown in Figure 16, the total displacement time histories are shown for Pier 6 at five different locations from bottom to top: footing, top of pier columns (east and west), and nodes of top truss chords (east and west). In addition, the total input displacements are also shown for reference. Both longitudinal and transverse displacements are shown. The relative displacement time histories for Pier 6 at the same five locations are shown in Figure 17. The relative displacement was derived by subtracting the support input displacement from the total displacement response. These time history plots show how the structural responses vary along the height of the pier and with time. The maximum values at various points on each pier are summarized in Table 6. At the roadway level (top of truss), maximum total displacements are: 29 inches longitudinal at Pier 4, and 23 inches transverse at Pier 10. At top of pier, the maximum displacements were 26 inches longitudinal and 18 inches transverse.

The maximum total transverse displacement at the midspan of the truss between Piers was 41 inches. However, the stress induced in the truss member was determined by the relative displacement which is much smaller. The transverse displacement responses relative to the deck at Pier 10 at several locations of Spans 7 and 8 are shown in Figure 18. The maximum relative transverse displacement at Span 8 was below 11 inches.

Hinge (Fixed and Expansion) Responses

The relative displacement responses between the two sides of each truss expansion hinges were calculated and are shown in Figure 19 for Hinge No. 1. Maximum openings were 26 inches at Hinge No. 1 adjacent to Pier 5 and 25 inches at Hinge No. 5 adjacent to Pier 9. Maximum forces that transferred through the fixed and expansion hinges and hangers are summarized in Table 7. The hinge details are shown in Figure 13. The hangers in the expansion hinge are made of 2 plates (24 x 1 1/2) for flanges and 1 plate (16 1/2 x 1) for the web. This is one of the most vulnerable components in the truss system. Demand-to-capacity ratios for these hangers under combined tension and bending are 2 to 3.

Truss Bearings

Time histories of bearing forces in all truss bearings were calculated. The maximum values are summarized in Table 8. Significant uplifting forces were induced in many bearings. These demands far exceed the available capacity of the existing bearings. Existing steel bearings are 2 to 3 feet high and consist of welded plates, stiffeners, pins, anchor bolts and bearing plates.
The limiting strength of each component was evaluated based on ultimate strength in shear, bearing and tension. Typically, the weakest link in the load transfer path provided a capacity almost an order of magnitude less than the calculated demand, i.e., demand/capacity ≥ 10.

Responses of the Pier and Shaft Substructure

One of the most critical factors in the seismic response evaluation is the idealization of the substructure-caisson-soil interaction. Detailed site response analyses were carried out to establish the effective linearized soil profiles at each pier location. Locations where soil liquefaction occurred under the Safety Evaluation earthquake input were identified. These factors were taken into account in the soil-structure interaction studies to determine the linearized foundation spring and damping coefficients. Given these boundary conditions, the earthquake demands on the piers were calculated. Maximum forces at the pier footings are summarized in Table 9.

Maximum forces at various elevations of the concrete shaft are calculated. The critical section in all piers are the column section at mid-level of the piers except for Piers 3 and 13 where the base sections are critical. The maximum demand/capacity ratios at critical sections are summarized in Table 10a. In both longitudinal and transverse directions, the governing demand/capacity ratio is caused by flexure at all pier locations.

Lateral Load Capacity of the Caisson Group

A detailed 3D nonlinear model was developed for Pier 6 which accounted for:

- Caisson tip uplifting.
- Nonlinear soil reaction on the caisson.
- Inelastic yielding of the concrete-filled steel caisson.

In this study, the concrete pier and footing are assumed to be elastic. A schematic of the model is shown in Figure 20 for Pier 6 which has ten caissons. Nonlinear static lateral load analyses were carried out by imposing incremental displacement at the top of the pier. Because of the high rigidity of the caissons, the pier capacity is not sensitive to the nonlinear soil capacity. The most critical factor is the uplifting capacity at the tip of the caisson or at the footing-caisson connection. For the caisson tip pinned condition (i.e., no uplift permitted), yielding of the caisson group occurs at a lateral load of 5400 kip in the transverse direction and 4500 kip in the longitudinal direction, respectively. These results are valid only if the caissons can resist the uplifting axial force.

The uplifting (tension) capacity of each caisson was estimated to be 1000 kip. If the caisson uplifting capacity was included in the model, the results for lateral load capacity are shown in Figure 21. For the transverse loading, the effect of uplifting on the lateral loading capacity is small due to the arrangement of the caissons in six rows. As shown in Figure 21f, five caissons reached uplift capacity first and then reached flexural yielding, provided that the flexural capacity will not be reduced as a consequence of the uplifting, i.e., tension failure. This may not be a conservative assumption.
In the longitudinal direction, the lateral capacity will be greatly reduced once uplift occurred as shown in Figures 21c and 21d. The lateral load capacity is reduced to about 2500 kip, and longitudinal instability of the pier may result.

If we define the yield point as the point in the action-deformation curve where significant nonlinearity occurs, the yield displacements and yield rotations can be derived from Figures 21a, b, c and d:

<table>
<thead>
<tr>
<th>Yield Displacement</th>
<th>Footing Level</th>
<th>Bearing Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse</td>
<td>13 inch</td>
<td>17 inch</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>7 inch</td>
<td>11 inch</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Yield Rotation</th>
<th>Footing Level</th>
<th>Bearing Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>About Longitudinal Axis</td>
<td>0.85 x 10^{-3} rad</td>
<td>-</td>
</tr>
<tr>
<td>About Transverse Axis</td>
<td>1.6 x 10^{-3} rad</td>
<td>-</td>
</tr>
</tbody>
</table>

At Pier 6 footing, the maximum displacements are 13 inches longitudinal and 17 inches transverse corresponding to displacement ductilities of 1.9 longitudinal and 1.3 transverse, respectively.

Another important factor in the load path is the footing caisson connection. It was estimated that the reinforcements in the footing concrete block can only transfer 50% of the plastic shear of the caisson.

**Truss Members and Connections**

Member forces for each truss member were calculated taking into account the secondary bending effect. The section properties have been calculated for all members and stored in a database. The stress checks were carried out in accordance with the AASHTO Load Factor criteria. For main truss members (chords, verticals and diagonals), the demand/capacity ratios are typically less than 2. Members in top and bottom laterals and transverse sway frames can be overstressed to a much higher level.

**SEISMIC PERFORMANCE UNDER FUNCTIONAL EVALUATION EARTHQUAKES**

As described earlier, the seismic performance of an important structure should be evaluated based on not only safety considerations but also functional considerations. In addition to the Safety Evaluation earthquake, two Function Evaluation events were established: a 100 year event and a 300 year event. The consideration of the 100 year event is due to the fact that a parallel new bridge may become on line in the next few years resulting in a redundant transportation route.
Based on results obtained for the Safety Evaluation earthquake, it is clear that the most vulnerable components in the load path are:

- Bearings,
- Concrete pier shaft, and
- Footing-caisson connections.

To provide a comparison, the seismic performance of the concrete pier shaft under the two functional evaluation events are described below:

Functional Evaluation Earthquake – 300 Year Return Period

Maximum forces at various locations on the concrete piers were calculated for the 300 year seismic event. None of the pier sections experienced axial tension during the seismic excitation. The maximum demand/capacity ratios for flexure and shear at critical sections are summarized in Table 10b. The deficiency of flexural moment about the transverse direction is greater than that of the shear deficiency. However, the margin is much smaller than previously reported for the Safety Evaluation event. In the longitudinal direction, the governing factor becomes shear. This is not desirable due to the brittle nature of shear failure.

Functional Evaluation Earthquake – 100 Year Return Period

For the 100 year event, the maximum seismic demand on the concrete shaft sections were calculated and the maximum demand/capacity ratios are summarized in Table 10c. In the longitudinal direction, the shear is the governing factor; in the transverse direction, both shear and flexure may govern.

SEISMIC VULNERABILITIES

Earthquake induced forces from bridge deck system are transferred to the foundation through the following path: floorbeam-truss connections, truss system and sway frames, hangers and connections at expansion hinges, bearings, concrete pier shafts, footing-caisson connections (shear and axial), steel caissons and caisson-foundation anchors in the rock. The weakest component governs the structure's strength, the potential failure mode and the serviceability loss of the bridge.

Truss Structure System

The seismic vulnerabilities of the truss structure are summarized below:

a. Floorbeam Connection to the Main Truss

The connection details are generally adequate for vertical and transverse earthquake loads. However, since the floorbeam is supported at the bottom, overturning of the floorbeam may occur as a result of the longitudinal earthquake loads.
b. Main Truss Members

In the main truss members (top and bottom chords, vertical and diagonal), the overstress ratio is typically low. The only exceptions are the top and bottom chords at the two panels near the support bearings.

c. Lateral Bracings and Transverse Sway Frames

All these bracing members are very weak due to the high slenderness ratio. Under compression loads, they will buckle and will result in the increase of the demand on the corresponding tension braces (by a factor of 2). The earthquake induced member forces are concentrated at the two panels on both sides of each bearing support.

d. Expansion Hinge Hangers of the Drop-in Spans

These members and connections are adequate for transferring vertical and longitudinal loads, but not sufficient for transferring large transverse earthquake loads.

e. Bearings

Shear capacity of bearings is limited to the strength of pintles, anchor bolts, pins, bearing plates, keeper plates, etc. The strength of some components is an order of magnitude less than the calculated elastic earthquake demand. The uplifting restraints of all bearings are small or do not exist. As a result, bearings may jump out of the bearing shoes.

Pier-Caisson Structure

The substructure of the truss span bridge is made of multiple-cell hollow concrete shaft supported on concrete footings and a group of eight or ten steel caissons. The critical sections were shown in Figure 22.

a. Concrete Pier Columns

The hollow concrete pier columns do not have sufficient flexural and shear capacities. Under earthquake loading in the transverse direction of the bridge, the pier columns in the upper sections will experience tension axial loads which result in very high flexural demand/capacity ratios. The acceptable demand/capacity ratios for these non-ductile hollow concrete sections are low. Unless we can improve the shear capacity as well as the flexural capacity, these sections will be the critical link in the load path.
b. Concrete Footings

The 25-foot thick multi-cell concrete footings serve as the load transfer from the piers down to the caissons. The reinforcements in the footing concrete block surrounding the embedded steel caissons are not sufficient to transfer the horizontal shear. The caisson is embedded 7 feet into the footing. The uplifting resistance is provided by the adhesion and friction between the steel shell and the seal concrete. The uplifting capacity of the footing is therefore quite low and will result in rocking response of the footing.

c. Steel Caissons

The 6-foot diameter steel caissons (with 1 inch wall thickness) are filled with concrete. These members were sunk into bedrock. The steel caisson is embedded about 2 feet into the bedrock. Sixty inch diameter "sockets" were drilled into the bedrock for a minimum length of 5 feet. The lower 20 feet of the caisson and the socket were reinforced as spiral concrete columns.

The axial force is transferred from the steel shell to the longitudinal reinforcements (15-#11 bars) of the spiral columns and to the bedrock through friction and end bearing. The critical link is the axial tension capacity of the spiral column which is only about 1000 kips.

The shear force is transferred through a combination of bearing of steel shell against the rock over the 2-foot embedment and the bending of the spiral column. There is some uncertainty on the steel shell bearing against the rock due to the method used in the initial construction. The flexural resistance of the spiral column will be greatly reduced if tension failure occurred. This combination of tension-flexural failure of the spiral column at the tip of the caisson is very critical to the integrity of the pier.

SEISMIC RETROFIT STUDY

Seismic resistance of the structure can be improved by providing:

- strength,
- ductility and energy dissipation, and
- seismic isolation.

Traditionally, the combination of strength and ductile energy dissipation is the ideal approach for better seismic performance. Due to the inherent deficiency in the existing piers, two retrofit strategies were developed. In the first strategy, the substructure and bearings are strengthened significantly to increase the capacities of the substructure. Supplemental damping is provided to the structure system by using friction dampers in the transverse sway frames to reduce the seismic demand.

In a second retrofit strategy, it was decided to explore the idea of hybrid sliding-restraining bearings. These are conventional low-profile bearings which are designed to slide at a certain horizontal force level and when the relative displacement exceeds a predetermined
level, additional restraint will be provided to limit the relative movement of the superstructure and the piers.

These studies were described in the report by Imbsen & Associates, Inc. (1993).

CONCLUSIONS AND RECOMMENDATIONS

A detailed seismic investigation of the Benicia-Martinez Bridge has been carried out using results from recent developments in spatially-varying ground motion. The seismic evaluation was conducted for the Safety Evaluation earthquake (1000 year return period) and two Function Evaluation earthquakes (300 year and 100 year return period).

The most critical components are truss bearings and non-ductile hollow concrete shaft piers. Under the safety evaluation earthquake, concrete piers are highly deficient in both flexural and shear. At the lower level events, even the flexural seismic demands were reduced; however, the seismic demands on shear were still high and became the governing factor. This shifting of governing failure mode (from flexure for the safety evaluation earthquake to shear for the function evaluation earthquake) dictates the priority and urgency of seismic retrofit.

Two retrofit strategies for main truss spans were proposed. Their seismic performances under a safety evaluation ground motion event were evaluated. Strategy No. 1 requires strengthening of the substructure to a higher capacity level. Supplemental damping introduced in the selected transverse sway frames was effective to reduce the transverse shear forces transmitted down to piers. However, the overall scheme is still essentially a strength approach. Strategy No. 2 utilizes conventional components, but allows the relative movement at bearings. However, energy dissipating seismic links and seismic stoppers are provided at each bearing to improve the seismic performance. This strategy gives designer better control of the seismic performance using the combination of seismic isolation, energy dissipation and displacement restraints.

Both strategies provide satisfactory seismic performance. However, Strategy No. 2 relies less on the strengthening of existing components which has always been a difficult and expensive task to accomplish. From this perspective, Strategy No. 2 is the preferred approach for the main truss spans.

Due to the highly uncertain nature of the seismic ground motion specifications and structural response prediction, it would be very important to assess the sensitivity of structural responses to the incoherent multiple seismic input. For this purpose, a multiple-support response spectrum method would be very desirable which accounts for various aspects of the spatially-varying ground motion, e.g. wave passage effect and incoherence. This method has been applied to the Golden Gate Bridge and compared reasonably well with the time history analysis result (Nakamura, et al 1993). The implementation of this method is being further refined and is currently being applied to the Vincent Thomas Bridge to establish further verification with time history analysis results (Liu, et al 1993).
Acknowledgments

This project was conducted by Imbsen & Associates, Inc. with the assistance of Geomatrix Consultants, International Civil Engineering Consultants and Faye Bernstein & Associates. Many individuals from IAI participated in the project. Their efforts are greatly appreciated. Geomatrix Consultants participated in providing the incoherency rock motions, developing the equivalent linear soil profiles at each pier locations, conducting lateral load capacity analysis of the caisson groups and evaluating the potential of soil liquefaction and deformation. International Civil Engineering Consultants (ICEC) conducted detailed soil-structure interaction analysis to develop foundation impedance matrices (stiffness and damping) as well as support input motion time histories at pier footings. ICEC also conducted a validation study for using the 3D soil-structure interaction methodology in the problem of soil and large-diameter caisson group interaction. Faye Bernstein & Associates assisted with the evaluation and concept retrofit design of the concrete pier shafts. The assistance provide by Caltrans personnel including, Mr. Stan Larsen and Mark Yashinsky are greatly appreciated.

References

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Geomatrix Consultants, Inc. (1992), Seismic Response for Proposed Benicia-Martinez Bridge Contra Costa and Solano Counties, Prepared for Caltrans, Division of Structures, Sacramento, California, December 1992


Caltrans (1992), Memo to Designers 20-4, Division of Structures, California Department of Transportation, September 1992


Table 1

Caltrans Division of Structures 3/9/93

SEISMIC PERFORMANCE CRITERIA
FOR THE DESIGN AND EVALUATION OF BRIDGES

<table>
<thead>
<tr>
<th>Ground Motion at Site</th>
<th>Minimum Performance Level</th>
<th>Important Bridge Performance Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Functional Evaluation</td>
<td>Immediate Service Level</td>
<td>Immediate Service Level</td>
</tr>
<tr>
<td></td>
<td>Repairable Damage</td>
<td>Minimal Damage</td>
</tr>
<tr>
<td>Safety Evaluation</td>
<td>Limited Service Level</td>
<td>Immediate Service Level</td>
</tr>
<tr>
<td></td>
<td>Significant Damage</td>
<td>Repairable Damage</td>
</tr>
</tbody>
</table>

DEFINITIONS

Immediate Service Level: Full access to normal traffic available almost immediately.

Limited Service Level: Limited access, (reduced lanes, light emergency traffic) possible within days.

Full service restorable within months.

Minimal Damage: Essentially elastic performance.

Repairable Damage: Damage that can be repaired with a minimum risk of losing functionality.

Significant Damage: A minimum risk of collapse, but damage that would require closure for repair.

Important Bridge (one or more of the following items present):

- Bridge required to provide secondary life safety. (Example: access to an emergency facility).
- Time for restoration of functionality after closure creates a major economic impact.
- Bridge formally designated as critical by a local emergency plan.

Safety Evaluation Ground Motion: Up to two methods may be used:

- Deterministically assessed ground motions from the maximum earthquake as defined by the Division of Mines and Geology Open-File Report 92-1 (1992).
- Probabilistically assessed ground motions with a long term return period (approx. 1000-2000 years).

For important bridges both methods shall be given consideration; however, the probabilistic evaluation shall be reviewed by a Caltrans approved consensus group. For all other bridges the motions shall be based only on the deterministic evaluation. In the future, the role of the two methods for other bridges shall be reviewed by a Caltrans approved consensus group.

Functional Evaluation Ground Motion:

Probabilistically assessed ground motions which have a 40% probability of occurring during the useful life of the bridge. The determination of this event shall be reviewed by a Caltrans approved consensus group. A separate Functional Evaluation if required only for Important Bridges. All other bridges are only required to meet specified design requirements to assure Minimum Functional Performance Level compliance.
Table 2: Ground Motion (Rock) Input Parameters

Peak Ground Acceleration, Velocity and Displacement and Duration of Strong Shaking of Modified Time Histories

Benicia-Martinez Bridge

<table>
<thead>
<tr>
<th>Component</th>
<th>Values for Modified Time Histories</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PGA (g)</td>
<td>PGV (cm/sec)</td>
<td>PGD (cm)</td>
<td>Duration (sec)</td>
</tr>
<tr>
<td>San Andreas Event (<em>M_w</em> = 8; <em>R</em> = 48 km)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H1</td>
<td>0.23</td>
<td>25.8</td>
<td>16.8</td>
<td>40</td>
</tr>
<tr>
<td>(Transverse)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H2</td>
<td>0.19</td>
<td>20.2</td>
<td>16.7</td>
<td>41</td>
</tr>
<tr>
<td>(Longitudinal)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V</td>
<td>0.15</td>
<td>20.0</td>
<td>21.7</td>
<td>29</td>
</tr>
<tr>
<td>Hayward Event (<em>M_w</em> = 7.4; <em>R</em> = 19 km)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H1</td>
<td>0.37</td>
<td>44.7</td>
<td>26.1</td>
<td>20</td>
</tr>
<tr>
<td>(Transverse)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H2</td>
<td>0.37</td>
<td>48.0</td>
<td>13.6</td>
<td>20</td>
</tr>
<tr>
<td>(Longitudinal)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V</td>
<td>0.30</td>
<td>19.7</td>
<td>10.3</td>
<td>25</td>
</tr>
<tr>
<td>Green Valley Event (<em>M_w</em> = 6.5; <em>R</em> = 3 km)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H1</td>
<td>0.91</td>
<td>101.8</td>
<td>32.9</td>
<td>11</td>
</tr>
<tr>
<td>(Transverse)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H2</td>
<td>0.91</td>
<td>82.9</td>
<td>20.4</td>
<td>11</td>
</tr>
<tr>
<td>(Longitudinal)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V</td>
<td>0.86</td>
<td>34.0</td>
<td>13.3</td>
<td>13</td>
</tr>
</tbody>
</table>

NOTES:

1. Duration refers to strong motion duration defined as time between 5 to 95 percent of energy measured by \( \int a^2(t)dt \).
2. Longitudinal refers to direction parallel to the alignment of the bridge. Transverse refers to direction perpendicular to the alignment of the bridge.
Table 3: Support Input Motion for Benicia-Martinez Bridge

<table>
<thead>
<tr>
<th>pier #</th>
<th>maximum displacement (ft)</th>
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Table 5: Quasi-Static Effect on Bearings and Hinges

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Table 6: Maximum Displacements at Piers

(a) Maximum Total Displacement Response of Piers

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<th>top of pier (west)</th>
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(b) Maximum Relative Displacement Response of Piers with Respect to Support Input Motions

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Table 7a: Maximum Hanger Forces (kips)

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Table 7b: Maximum Forces in Fixed and Expansion Hinges (kips)

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### Table 8: Maximum Bearing Forces at Pier Top

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<th>East (kips)</th>
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### Table 9: Maximum Pier Forces at Base

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Table 10: Seismic Vulnerability of the Hollow Concrete Shaft - Maximum Demand/Capacity Ratios for Flexural and Shear

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* Section failed in tension resulting in very low capacity.

b. 300 Year Functional Evaluation Earthquake

<table>
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c. 100 Year Functional Evaluation Earthquake

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M<sub>L</sub>, M<sub>T</sub> = Moment about longitudinal and transverse axis, respectively.
V<sub>L</sub>, V<sub>T</sub> = Longitudinal and transverse shear, respectively.
Figure 2: Benicia-Martinez Bridge Soil Profile
### Table

<table>
<thead>
<tr>
<th>Pier No.</th>
<th>A (ave.) (ft)</th>
<th>B (ave.) (ft)</th>
<th>C (ave.) (ft)</th>
<th>Ave. Caisson Bedrock to Bot. Ftg. (ft)</th>
<th>D (ft)</th>
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</table>

- **A** = 'Socket' depth
- **B** = Overburden depth
- **C** = Water depth at Mean Sea Level

### Diagram and Text

**Figure 3: Typical Piers**

**Plan of Footing**

**Piers 4 thru 12**

Omit these caissons at Piers 4 & 12
Figure 4: Target Response Spectra for Three Earthquake Sources - Horizontal Motions (84th Percentile Spectrum)

Figure 5: Equal-Hazard Response Spectra for the Benicia-Martinez Bridge Site (5% Damping)
Figure 6: Typical Piers - Benicia-Martinez Bridge
Figure 7: Free Field Site Response Analysis

Figure 8: Foundation Impedance Soil-Structure Interaction Analysis
Figure 12: Benicia-Martinez Bridge - Details of Fixed and Expansion Hinges
Figure 13: Longitudinal Vibration Mode - Benicia-Martinez Bridge

Mode # 6 2.08 Sec

Figure 14: Transverse Vibration Mode - Benicia-Martinez Bridge

Mode # 5 2.13 Sec
Relative Transverse Displacement (Bridge Deck)

Figure 17: Relative Transverse Displacement Time Histories of the Deck (with respect to top of pier)

Expansion Hinge 5 Opening

Figure 18: Relative Displacement Responses of Expansion Hinge No. 5
Figure 19: Pier-Caisson Model (Pier 6)
Figure 20: Pier 6 Lateral Load Analysis - Caisson Uplift Capacity Limited to 1000 kips

a) Transverse Shear Capacity
b) Moment-Rotational Curve about Longitudinal Axis

c) Longitudinal Shear Capacity
d) Moment-Rotational Curve about Transverse Axis
Moment (trans.-axis)—Axial Force Interaction
Caisson Shaft Top Section of Pier 6
(caisson tip tension limited to 1000 kips)

e) Load Path under Longitudinal Load

Moment (long.-axis)—Axial Force Interaction
Caisson Shaft Top Section of Pier 6
(caisson tip tension limited to 1000 kips)
f) Load Path under Transverse Load

Figure 20 continued
Appendix A: ATC-35 Seminar Programs

**LOS ANGELES SEMINAR**
January 26, 1994
Los Angeles Hilton and Towers
Los Angeles, California

<table>
<thead>
<tr>
<th>Time</th>
<th>Session</th>
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<tbody>
<tr>
<td>8:15 a.m.</td>
<td>Seminar Registration</td>
</tr>
<tr>
<td>9:00 a.m.</td>
<td>Introductory Comments</td>
</tr>
<tr>
<td><strong>PART 1: REGIONAL EARTHQUAKE RISK</strong></td>
<td></td>
</tr>
<tr>
<td>9:15-9:55 a.m.</td>
<td>New Knowledge of Southern California Earthquake Potential</td>
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<tr>
<td></td>
<td>Lucile Jones, U. S. Geological Survey, Pasadena, California</td>
</tr>
<tr>
<td>9:55-10:25 a.m.</td>
<td>New Knowledge of Ground Motion Attenuation</td>
</tr>
<tr>
<td>10:25-10:45 a.m.</td>
<td>Coffee Break</td>
</tr>
<tr>
<td>10:45-11:15 a.m.</td>
<td>Ground Motion Mapping—Past, Present and Future</td>
</tr>
<tr>
<td><strong>PART 2: STRONG GROUND MOTION ESTIMATION</strong></td>
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</tr>
<tr>
<td>11:15-11:45 a.m.</td>
<td>Fundamentals of the Generation and Propagation of Seismic Waves</td>
</tr>
<tr>
<td></td>
<td>Thomas Heaton, U. S. Geological Survey, Pasadena, California</td>
</tr>
<tr>
<td>11:45-12:15 p.m.</td>
<td>New Developments in Estimating Near-Source Ground Motions</td>
</tr>
<tr>
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<td>John Boatwright, U. S. Geological Survey, Menlo Park, California</td>
</tr>
<tr>
<td>12:15-1:15 p.m.</td>
<td>Buffet Lunch</td>
</tr>
<tr>
<td>1:15-1:45 p.m.</td>
<td>New Developments in Estimating Site Response Effects on Ground Motions</td>
</tr>
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<td>Roger Borcherdt, U. S. Geological Survey, Menlo Park, California</td>
</tr>
<tr>
<td>1:45-2:15 p.m.</td>
<td>New Developments in Estimating Basin Response Effects on Ground Motions</td>
</tr>
<tr>
<td>2:15-2:45 p.m.</td>
<td>New Developments in Estimating Spatial Variations and Incoherence of Ground Motions</td>
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<tr>
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<td>Paul Spudich, U. S. Geological Survey, Menlo Park, California</td>
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<tr>
<td>2:45-3:05 p.m.</td>
<td>Coffee Break</td>
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<tr>
<td><strong>PART 3: IMPLICATIONS OF NEW KNOWLEDGE AND NEW DEVELOPMENTS FOR ENGINEERING DESIGN PRACTICE</strong></td>
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<tr>
<td>3:05-3:35 p.m.</td>
<td>Implications for Site-Specific Ground Motion Estimation</td>
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<td>Yoshiharu Moriwaki, Woodward-Clyde Consultants, Santa Ana, California</td>
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<tr>
<td>3:35-4:05 p.m.</td>
<td>Implications for Seismic Design of Buildings</td>
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<tr>
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<td>Chris Poland, H. J. Degenkolb Associates, San Francisco, California</td>
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<tr>
<td>4:05-4:35 p.m.</td>
<td>Implications for Seismic Design of Bridges</td>
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<td>Roy Imbsen, Imbsen &amp; Associates, Sacramento, California</td>
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<tr>
<td>4:35-5:30 p.m.</td>
<td>Panel and Audience Discussion</td>
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<td>Moderator, Charles Thiel, Telesis Consultants, Piedmont, California</td>
</tr>
<tr>
<td></td>
<td>Greg Brandow, Brandow &amp; Johnson Marsh, Law/Crandall, Inc.</td>
</tr>
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<td>Trailer Martin, John A. Martin &amp; Assoc. Brittain Poteet, Gannett Fleeming, Inc.</td>
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<td></td>
<td>Wolfgang Roth, Dames &amp; Moore</td>
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SAN FRANCISCO SEMINAR PROGRAM  
January 27, 1994  
Pan Pacific Hotel  
San Francisco, California

<table>
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<td>8:15 a.m.</td>
<td>Seminar Registration</td>
</tr>
<tr>
<td>9:00 a.m.</td>
<td>Introductory Comments</td>
</tr>
<tr>
<td>9:15-9:55 a.m.</td>
<td><strong>PART 1: REGIONAL EARTHQUAKE RISK</strong></td>
</tr>
<tr>
<td>9:15-9:55 a.m.</td>
<td>New Knowledge of Northern California Earthquake Potential</td>
</tr>
<tr>
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<td>David Schwartz, U. S. Geological Survey, Menlo Park, California</td>
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<tr>
<td>9:55-10:25 a.m.</td>
<td>New Knowledge of Ground Motion Attenuation</td>
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<td>10:25-10:45 a.m.</td>
<td>Coffee Break</td>
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<tr>
<td>10:45-11:15 a.m.</td>
<td><strong>PART 2: STRONG GROUND MOTION ESTIMATION</strong></td>
</tr>
<tr>
<td></td>
<td>Ground Motion Mapping — Past, Present and Future</td>
</tr>
<tr>
<td>11:15-11:45 a.m.</td>
<td>Fundamentals of the Generation and Propagation of Seismic Waves</td>
</tr>
<tr>
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<td>Thomas Heaton, U. S. Geological Survey, Pasadena, California</td>
</tr>
<tr>
<td>11:45-12:15 p.m.</td>
<td>New Developments in Estimating Near-Source Ground Motions</td>
</tr>
<tr>
<td></td>
<td>John Boatwright, U. S. Geological Survey, Menlo Park, California</td>
</tr>
<tr>
<td>12:15-1:15 p.m.</td>
<td>Buffet Lunch</td>
</tr>
<tr>
<td>1:15-1:45 p.m.</td>
<td>New Developments in Estimating Site Response Effects on Ground Motions</td>
</tr>
<tr>
<td></td>
<td>Roger Borcherdt, U. S. Geological Survey, Menlo Park, California</td>
</tr>
<tr>
<td>1:45-2:15 p.m.</td>
<td>New Developments in Estimating Basin Response Effects on Ground Motions</td>
</tr>
<tr>
<td>2:15-2:45 p.m.</td>
<td>New Developments in Estimating Spatial Variations and Incoherence of Ground Motions</td>
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<td>Paul Spudich, U. S. Geological Survey, Menlo Park, California</td>
</tr>
<tr>
<td>2:45-3:05 p.m.</td>
<td>Coffee Break</td>
</tr>
<tr>
<td>3:05-3:35 p.m.</td>
<td><strong>PART 3: IMPLICATIONS OF NEW KNOWLEDGE AND NEW DEVELOPMENTS FOR ENGINEERING DESIGN PRACTICE</strong></td>
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<td></td>
<td>Implications for Site-Specific Ground Motion Estimation</td>
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<td>Maurice Power, Geomatrix Consultants, San Francisco, California</td>
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<tr>
<td>3:35-4:05 p.m.</td>
<td>Implications for Seismic Design of Buildings</td>
</tr>
<tr>
<td></td>
<td>Chris Poland, H. J. Degenkolb Associates, San Francisco, California</td>
</tr>
<tr>
<td>4:05-4:35 p.m.</td>
<td>Implications for Seismic Design of Bridges</td>
</tr>
<tr>
<td></td>
<td>Roy Imbsen, Imbsen &amp; Associates, Sacramento, California</td>
</tr>
<tr>
<td>4:35-5:30 p.m.</td>
<td>Panel and Audience Discussion</td>
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<td>Moderator, Charles Thiel, Telesis Consultants, Piedmont, California</td>
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<td>Ronald Gallagher, R. P. Gallagher Assoc.</td>
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<td>Lelio Mejia, Woodward-Clyde Consultants</td>
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<td>Mark Saunders, Rutherford &amp; Chekene</td>
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SEATTLE SEMINAR PROGRAM
February 2, 1994
Marriott Sea-Tac Airport Hotel
Seattle, Washington

8:15 a.m. Seminar Registration
9:00 a.m. Introductory Comments

PART 1: REGIONAL EARTHQUAKE RISK
9:15-9:55 a.m.
New Knowledge of Pacific Northwest Earthquake Potential
Craig Weaver, U. S. Geological Survey, Seattle, Washington
9:55-10:25 a.m.
New Knowledge of Ground Motion Attenuation
David Boore, U. S. Geological Survey, Menlo Park, California
10:25-10:45 a.m. Coffee Break
10:45-11:15 a.m.
Ground Motion Mapping—Past, Present and Future
11:15-11:45 a.m.
Fundamentals of the Generation and Propagation of Seismic Waves
Thomas Heaton, U. S. Geological Survey, Pasadena, California
11:45-12:15 p.m.
New Developments in Estimating Near-Source Ground Motions
John Boatwright, U. S. Geological Survey, Menlo Park, California
12:15-1:15 p.m. Buffet Lunch
1:15-1:45 p.m.
New Developments in Estimating Site Response Effects on Ground Motions
Roger Borcherdt, U. S. Geological Survey, Menlo Park, California
1:45-2:15 p.m.
New Developments in Estimating Basin Response Effects on Ground Motions
2:15-2:45 p.m.
New Developments in Estimating Spatial Variations and Incoherence of Ground Motions
Paul Spudich, U. S. Geological Survey, Menlo Park, California
2:45-3:05 p.m. Coffee Break

PART 2: STRONG GROUND MOTION ESTIMATION
3:05-3:35 p.m.
Implications for Site-Specific Ground Motion Estimation
C. B. Crouse, Dames & Moore, Seattle, Washington
3:35-4:05 p.m.
Implications for Seismic Design of Buildings
Chris Poland, H. J. Degenkolb Associates, San Francisco, California
4:05-4:35 p.m.
Implications for Seismic Design of Bridges
Roy Imbsen, Imbsen & Associates, Sacramento, California
4:35-5:30 p.m.
Panel and Audience Discussion
Moderator, Charles Thiel, Telesis Consultants, Piedmont, California
James Carpenter, Bruce Olsen Cons. Engr.
John Clark, Anderson Bjornstad Kane Jacobs
Paul Grant, Shannon & Wilson, Inc.
John Hooper, Ratti Swenson Perbix, Inc.
James Thompson, Geo Engineers
NEW YORK SEMINAR PROGRAM  
February 9, 1994  
Marriott Marquis Hotel  
New York, New York

<table>
<thead>
<tr>
<th>Time</th>
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<tbody>
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<td></td>
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<td></td>
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<tr>
<td>9:15-9:55 a.m.</td>
<td><strong>PART 1: REGIONAL EARTHQUAKE RISK</strong></td>
<td></td>
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<tr>
<td>10:25-10:45 a.m.</td>
<td>Coffee Break</td>
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<tr>
<td>11:15-11:45 a.m.</td>
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<td></td>
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<tr>
<td>12:15-1:15 p.m.</td>
<td>Buffet Lunch</td>
<td></td>
</tr>
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<td>1:45-2:15 p.m.</td>
<td>New Developments in Estimating Basin Response Effects on Ground Motions</td>
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<td>2:45-3:05 p.m.</td>
<td>Coffee Break</td>
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<td>3:05-3:35 p.m.</td>
<td>Implications of Site-Specific Ground Motion Estimation</td>
<td>Maurice Power, Geomatrix Consultants, San Francisco, California</td>
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<td>3:35-4:05 p.m.</td>
<td>Implications for Seismic Design of Buildings</td>
<td>Chris Poland, H. J. Degenkolb Associates, San Francisco, California</td>
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<td>4:05-4:35 p.m.</td>
<td>Implications for Seismic Design of Bridges</td>
<td>Roy Imbsen, Imbsen &amp; Associates, Sacramento, California</td>
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<td>4:35-5:30 p.m.</td>
<td>Panel and Audience Discussion</td>
<td>Serafin Arzoumanidis, Steinman Engineers</td>
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<td>Moderator, Charles Thiel, Telesis Consulting Engineers</td>
<td>Peter Edinger, Meuser Rutridge Consulting Engineers</td>
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<td>Jacob Grossman, Rosenwasser/Grossman Consulting Engineers</td>
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<td>Klaus Jacob, Lamont-Doherty Earth Observatory</td>
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<td>Joseph Kelly, Port Authority of New York and New Jersey</td>
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<td>Guy Nordenson, Ove Arup &amp; Partners</td>
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MEMPHIS SEMINAR PROGRAM
February 10, 1994
The Peabody Hotel
Memphis, Tennessee

8:15 a.m. Seminar Registration
9:00 a.m. Introductory Comments

PART 1: REGIONAL EARTHQUAKE RISK
9:15-9:55 a.m.
New Knowledge of Central United States Earthquake Potential
Eugene Schweig, U. S. Geological Survey and Center for Earthquake Research and Information, Memphis, Tennessee

9:55-10:25 a.m.
New Knowledge of Ground Motion Attenuation
David Boore, U. S. Geological Survey, Menlo Park, California

10:25-10:45 a.m. Coffee Break

10:45-11:15 a.m.
Ground Motion Mapping—Past, Present and Future

11:15-11:45 a.m.
Fundamentals of the Generation and Propagation of Seismic Waves
Thomas Heaton, U. S. Geological Survey, Pasadena, California

11:45-12:15 p.m.
New Developments in Estimating Near-Source Ground Motions
John Boatwright, U. S. Geological Survey, Menlo Park, California

12:15-1:15 p.m. Buffet Lunch

1:15-1:45 p.m.
New Developments in Estimating Site Response Effects on Ground Motions
Roger Borchert, U. S. Geological Survey, Menlo Park, California

1:45-2:15 p.m.
New Developments in Estimating Basin Response Effects on Ground Motions

2:15-2:45 p.m.
New Developments in Estimating Spatial Variations and Incoherence of Ground Motions
Paul Spudich, U. S. Geological Survey, Menlo Park, California

2:45-3:05 p.m. Coffee Break

PART 2: STRONGGROUND MOTION ESTIMATION
11:15-11:45 a.m.
Implications for Site-Specific Ground Motion Estimation
Maurice Power, Geomatrix Consultants, San Francisco, California

11:45-12:15 p.m.
Implications for Seismic Design of Buildings
Chris Poland, H. J. Degenkolb Associates, San Francisco, California

12:15-1:15 p.m. Buffet Lunch

1:15-1:45 p.m.
Implications for Seismic Design of Bridges
Roy Imbsen, Imbsen & Associates, Sacramento, California

3:05-3:35 p.m.
Panel and Audience Discussion
Moderator, Charles Thiel, Telesis Consultants, Piedmont, California
Thomas Cooling, Woodward-Clyde
Warner Howe, Consulting Engineer
Terry Hughes, Shelby County and Memphis Building Official
Paul Murray, Lindsey and Associates
Ted Preuss, Theiss Engineers
Appendix B: ATC-35 Project Participants

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Los Angeles, California 90017
One of the primary purposes of Applied Technology Council is to develop resource documents that translate and summarize useful information to practicing engineers. This includes the development of guidelines and manuals, as well as the development of research recommendations for specific areas determined by the profession. ATC is not a code development organization, although several of the ATC project reports serve as resource documents for the development of codes, standards and specifications.

Applied Technology Council conducts projects that meet the following criteria:

1. The primary audience or benefactor is the design practitioner in structural engineering.

2. A cross section or consensus of engineering opinion is required to be obtained and presented by a neutral source.

3. ATC is requested to conduct the project by the project sponsor.

A brief description of several major completed projects and reports is given in the following section. Funding for projects is obtained from government agencies and tax-deductible contributions from the private sector.

ATC-1: This project resulted in five papers that were published as part of Building Practices for Disaster Mitigation, Building Science Series 46, proceedings of a workshop sponsored by the National Science Foundation (NSF) and the National Bureau of Standards (NBS). Available through the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, VA 22151, as NTIS report No. COM-73-50188.

ATC-2: The report, An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings, was funded by NSF and NBS and was conducted as part of the Cooperative Federal Program in Building

ATC-3: The report, Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC-3-06), was funded by NSF and NBS. The second printing of this report, which included proposed amendments, is available through the ATC office. (505 pages plus proposed amendments)

Abstract: The tentative provisions in this document represent the result of a concerted effort by a multi-disciplinary team of 85 nationally recognized experts in earthquake engineering. The project involved representation from all sections of the United States and had wide review by affected building industry and regulatory groups. The provisions embodied several new concepts that were significant departures from existing seismic design provisions. The second printing of this document contains proposed amendments prepared by a joint committee of the Building Seismic Safety Council (BSSC) and the NBS; the proposed amendments were published separately by BSSC and NBS in 1982.

ATC-3-2: The project, Comparative Test Designs of Buildings Using ATC-3-06 Tentative Provisions, was funded by NSF. The project consisted of a study to develop and plan a program for making comparative test designs of the ATC-3-06 Tentative Provisions. The project report was written to be used by
the Building Seismic Safety Council in its refinement of the ATC-3-06 Tentative Provisions.

ATC-3-4: The report, Redesign of Three Multistory Buildings: A Comparison Using ATC-3-06 and 1982 Uniform Building Code Design Provisions, was published under a grant from NSF. Available through the ATC office. (112 pages)

Abstract: This report evaluates the cost and technical impact of using the 1978 ATC-3-06 report, Tentative Provisions for the Development of Seismic Regulations for Buildings, as amended by a joint committee of the Building Seismic Safety Council and the National Bureau of Standards in 1982. The evaluations are based on studies of three existing California buildings redesigned in accordance with the ATC-3-06 Tentative Provisions and the 1982 Uniform Building Code. Included in the report are recommendations to code implementing bodies.

ATC-3-5: This project, Assistance for First Phase of ATC-3-06 Trial Design Program Being Conducted by the Building Seismic Safety Council, was funded by the Buildings Seismic Safety Council and provided the services of the ATC Senior Consultant and other ATC personnel to assist the BSSC in the conduct of the first phase of its Trial Design Program. The first phase provided for trial designs conducted for buildings in Los Angeles, Seattle, Phoenix, and Memphis.

ATC-3-6: This project, Assistance for Second Phase of ATC-3-06 Trial Design Program Being Conducted by the Building Seismic Safety Council, was funded by the Building Seismic Safety Council and provided the services of the ATC Senior Consultant and other ATC personnel to assist the BSSC in the conduct of the second phase of its Trial Design Program. The second phase provided for trial designs conducted for buildings in New York, Chicago, St. Louis, Charleston, and Fort Worth.

ATC-4: The report, A Methodology for Seismic Design and Construction of Single-Family Dwellings, was published under a contract with the Department of Housing and Urban Development (HUD). Available through the ATC office. (576 pages)

Abstract: This report presents the results of an in-depth effort to develop design and construction details for single-family residences that minimize the potential economic loss and life-loss risk associated with earthquakes. The report: (1) discussed the ways structures behave when subjected to seismic forces, (2) sets forth suggested design criteria for conventional layouts of dwellings constructed with conventional materials, (3) presents construction details that do not require the designer to perform analytical calculations, (4) suggests procedures for efficient plan-checking, and (5) presents recommendations including details and schedules for use in the field by construction personnel and building inspectors.

ATC-4-1: The report, The Home Builders Guide for Earthquake Design, was published under a contract with HUD. Available through the ATC office. (57 pages)

Abstract: This report is a 57-page abridged version of the ATC-4 report. The concise, easily understood text of the Guide is supplemented with illustrations and 46 construction details. The details are provided to ensure that houses contain structural features that are properly positioned, dimensioned and constructed to resist earthquake forces. A brief description is included on how earthquake forces impact on houses and some precautionary constraints are given with respect to site selection and architectural designs.

ATC-5: The report, Guidelines for Seismic Design and Construction of Single-Story Masonry Dwellings in Seismic Zone 2, was developed under a contract with HUD. Available through the ATC office. (38 pages)

Abstract: The report offers a concise methodology for the earthquake design
and construction of single-story masonry
dwellings in Seismic Zone 2 of the
United States, as defined by the 1973
Uniform Building Code. The guidelines
are based in part on shaking table tests
of masonry construction conducted at
the University of California at Berkeley
Earthquake Engineering Research
Center. The report is written in simple
language and includes basic house plans,
wall evaluations, detail drawings, and
material specifications.

ATC-6: The report, *Seismic Design Guidelines
for Highway Bridges*, was published under a
contract with the Federal Highway
Administration (FHWA). Available through
the ATC office. (210 pages)

Abstract: The Guidelines are the
recommendations of a team of sixteen
nationally recognized experts that
included consulting engineers,
academics, state highway engineers, and
federal agency representatives. The
Guidelines, applicable for use in all
parts of the U.S., include a preliminary
designing roof and floor systems so
ditches can function as horizontal
diaphragms in a lateral force resisting
system. Analytical procedures,
connection details and design examples
are included in the Guidelines.

ATC-7: The report, *Guidelines for the Design
of Horizontal Wood Diaphragms*, was
published under a grant from NSF. Available
through the ATC office. (190 pages)

ATC-7-1: The report, *Proceedings of a
Workshop on Design of Horizontal Wood
Diaphragms*, was published under a grant from
NSF. Available through the ATC office. (302
pages)

ATC-7-2: The report, *Seismic Retrofitting
Guidelines for Highway Bridges*, was published
under a contract with FHWA. Available
through the ATC office. (220 pages)

Abstract: The Guidelines are the
recommendations of a team of thirteen
nationally recognized experts that
included consulting engineers,
academics, state highway engineers, and
federal agency representatives. The
Guidelines, applicable for use in all
parts of the U.S., include a preliminary
screening procedure, methods for
evaluating an existing bridge in detail,
and potential retrofitting measures for
the most common bridge deficiencies.
Also included are special design
requirements for various retrofitting
measures.

ATC-8: This report, *Proceedings of a
Workshop on the Design of Prefabricated
Concrete Buildings for Earthquake Loads*, was
funded by NSF. Available through the ATC
office. (400 pages)

Abstract: The report includes seven
papers on state-of-the-practice and two
papers on recent research. Also
included are recommendations for
future research that were developed by
the 35 participants.
Abstract: The report includes eighteen state-of-the-art papers and six summary papers. Also included are recommendations for future research that were developed by the 43 workshop participants.

ATC-9: The report, *An Evaluation of the Imperial County Services Building Earthquake Response and Associated Damage*, was published under a grant from NSF. Available through the ATC office. (231 pages)

Abstract: The report presents the results of an in-depth evaluation of the Imperial County Services Building, a 6-story reinforced concrete frame and shear wall building severely damaged by the October 15, 1979 Imperial Valley, California, earthquake. The report contains a review and evaluation of earthquake damage to the building; a review and evaluation of the seismic design; a comparison of the requirements of various building codes as they relate to the building; and conclusions and recommendations pertaining to future building code provisions and future research needs.

ATC-10: This report, *An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance*, was funded by the U.S. Geological Survey (USGS). Available through the ATC office. (114 pages)

Abstract: The report contains an in-depth analytical evaluation of the ultimate or limit capacity of selected representative building framing types, a discussion of the factors affecting the seismic performance of buildings, and a summary and comparison of seismic design and seismic risk parameters currently in widespread use.

ATC-10-1: This report, *Critical Aspects of Earthquake Ground Motion and Building Damage Potential*, was co-funded by the USGS and the NSF. Available through the ATC office. (259 pages)

Abstract: This document contains 19 state-of-the-art papers on ground motion, structural response, and structural design issues presented by prominent engineers and earth scientists in an ATC seminar. The main theme of the papers is to identify the critical aspects of ground motion and building performance that currently are not being considered in building design. The report also contains conclusions and recommendations of working groups convened after the Seminar.

ATC-11: The report, *Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers*, was published under a grant from NSF. Available through the ATC office. (184 pages)

Abstract: This document presents the results of an in-depth review and synthesis of research reports pertaining to cyclic loading of reinforced concrete shear walls and cyclic loading of joint reinforced concrete frames. More than 125 research reports published since 1971 are reviewed and evaluated in this report. The preparation of the report included a consensus process involving numerous experienced design professionals from throughout the United States. The report contains reviews of current and past design practices, summaries of research developments, and in-depth discussions of design implications of recent research results.

ATC-12: This report, *Comparison of United States and New Zealand Seismic Design Practices for Highway Bridges*, was published under a grant from NSF. Available through the ATC office. (270 pages)

Abstract: The report contains summaries of all aspects and innovative design procedures used in New Zealand as well as comparison of United States and New Zealand design practice. Also included are research recommendations developed at a 3-day workshop in New Zealand attended by 16 U.S. and 5 New Zealand bridge design engineers and researchers.
ATC-12: This report, *Proceedings of Second Joint U.S.-New Zealand Workshop on Seismic Resistance of Highway Bridges*, was published under a grant from NSF. Available through the ATC office. (272 pages)

Abstract: This report contains written versions of the papers presented at this 1985 Workshop as well as a list and prioritization of workshop recommendations. Included are summaries of research projects currently being conducted in both countries as well as state-of-the-practice papers on various aspects of design practice. Topics discussed include bridge design philosophy and loadings; design of columns, footings, piles, abutments and retaining structures; geotechnical aspects of foundation design; seismic analysis techniques; seismic retrofitting; case studies using base isolation; strong-motion data acquisition and interpretation; and testing of bridge components and bridge systems.

ATC-13: The report, *Earthquake Damage Evaluation Data for California*, was developed under a contract with the Federal Emergency Management Agency (FEMA). Available through the ATC office. (492 pages)

Abstract: This report presents expert-opinion earthquake damage and loss estimates for existing industrial, commercial, residential, utility and transportation facilities in California. Included are damage probability matrices for 78 classes of structures and estimates of time required to restore damaged facilities to pre-earthquake usability. The report also describes the inventory information essential for estimating economic losses and the methodology used to develop the required data.

ATC-14: The report, *Evaluating the Seismic Resistance of Existing Buildings*, was developed under a grant from the NSF. Available through the ATC office. (370 pages)

Abstract: This report, written for practicing structural engineers, describes a methodology for performing preliminary and detailed building seismic evaluations. The report contains a state-of-practice review; seismic loading criteria; data collection procedures; a detailed description of the building classification system; preliminary and detailed analysis procedures; and example case studies, including non-structural considerations.

ATC-15: This report, *Comparison of Seismic Design Practices in the United States and Japan*, was published under a grant from NSF. Available through the ATC office. (317 pages)

Abstract: The report contains detailed technical papers describing current design practices in the United States and Japan as well as recommendations emanating from a joint U.S.-Japan workshop held in Hawaii in March, 1984. Included are detailed descriptions of new seismic design methods for buildings in Japan and case studies of the design of specific buildings (in both countries). The report also contains an overview of the history and objectives of the Japan Structural Consultants Association.

ATC-15-1: The report, *Proceedings of Second U.S.-Japan Workshop on Improvement of Building Seismic Design and Construction Practices*, was published under a grant from NSF. Available through the ATC office. (412 pages)

Abstract: This report contains 23 technical papers presented at this San Francisco workshop in August, 1986, by practitioners and researchers from the U.S. and Japan. Included are state-of-the-practice papers and case studies of actual building designs and information on regulatory, contractual, and licensing issues.

ATC-15-2: The report, *Proceedings of Third U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, was published jointly by ATC and...
the Japan Structural Consultants Association. Available through the ATC office. (358 pages)

Abstract: This report contains 21 technical papers presented at this Tokyo, Japan, workshop in July, 1988, by practitioners and researchers from the U.S., Japan, China, and New Zealand. Included are state-of-the-practice papers on various topics, including braced steel frame buildings, beam-column joints in reinforced concrete buildings, summaries of comparative U.S. and Japanese design, and base isolation and passive energy dissipation devices.


Abstract: This report contains 22 technical papers presented at this Kailua-Kona, Hawaii, workshop in August, 1990 by practitioners and researchers from the U.S., Japan, and Peru. Included are papers on postearthquake building damage assessment, acceptable earthquake damage, repair and retrofit of earthquake damaged buildings, base-isolated buildings, including Architectural Institute of Japan recommendations for design, active damping systems, wind-resistant design, and summaries of working group conclusions and recommendations.

ATC-16: This project, Development of a 5-Year Plan for Reducing the Earthquake Hazards Posed by Existing Nonfederal Buildings, was funded by FEMA and was conducted by a joint venture of ATC, the Building Seismic Safety Council and the Earthquake Engineering Research Institute. The project involved a workshop in Phoenix, Arizona, where approximately 50 earthquake specialists met to identify the major tasks and goals for reducing the earthquake hazards posed by existing nonfederal buildings nationwide. The plan was developed on the basis of nine issue papers presented at the workshop and workshop working group discussions. *The Workshop Proceedings and Five-Year Plan* are available through the Federal Emergency Management Agency, 500 "C" Street, S.W., Washington, DC 20472.

ATC-17: This report, *Proceedings of a Seminar and Workshop on Base Isolation and Passive Energy Dissipation*, was published under a grant from NSF. Available through the ATC office. (478 pages)

Abstract: The report contains 42 papers describing the state-of-the-art and state-of-the-practice in base-isolation and passive energy-dissipation technology. Included are papers describing case studies in the United States, applications and developments worldwide, recent innovations in technology development, and structural and ground motion issues. Also included is a proposed 5-year research agenda that addresses the following specific issues: (1) strong ground motion; (2) design criteria; (3) materials, quality control, and long-term reliability; (4) life cycle cost methodology; and (5) system response.

ATC-20: The report, *Procedures for Postearthquake Safety Evaluation of Buildings*, was developed under a contract from the California Office of Emergency Services (OES), California Office of Statewide Health Planning and Development (OSHPD) and FEMA. Available through the ATC office (152 pages)

Abstract: This report provides procedures and guidelines for making on-the-spot evaluations and decisions regarding continued use and occupancy of earthquake damaged buildings. Written specifically for volunteer structural engineers and building inspectors, the report includes rapid and detailed evaluation procedures for inspecting buildings and posting them as "inspected" (apparently safe), "limited entry" or "unsafe". Also included are special procedures for evaluation of essential buildings (e.g., hospitals), and
ATC-20-1: The report, *Field Manual: Postearthquake Safety Evaluation of Buildings*, was developed under a contract from OES and OSHPD. Available through the ATC office (114 pages)

Abstract: This report, a companion Field Manual for the ATC-20 report, summarizes the postearthquake safety evaluation procedures in brief concise format designed for ease of use in the field.

ATC-21: The report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, was developed under a contract from FEMA. Available through the ATC office. (185 pages)

Abstract: This report describes a rapid visual screening procedure for identifying those buildings that might pose serious risk of loss of life and injury, or of severe curtailment of community services, in case of a damaging earthquake. The screening procedure utilizes a methodology based on a "sidewalk survey" approach that involves identification of the primary structural load resisting system and building materials, and assignment of a basic structural hazards score and performance modification factors based on observed defects. Application of the methodology identifies those buildings that are potentially hazardous and should be analyzed in more detail by a professional engineer experienced in seismic design.

ATC-21-1: The report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation*, was developed under a contract from FEMA. Available through the ATC office. (137 pages)

Abstract: Included in this report are (1) a review and evaluation of existing procedures; (2) a listing of attributes considered ideal for a rapid visual screening procedures; and (3) a technical discussion of the recommended rapid visual screening procedure that is documented in the ATC-21 report.

ATC-21-2: The report, *Earthquake Damaged Buildings: An Overview of Heavy Debris and Victim Extrication*, was developed under a contract from FEMA. Available through the ATC office. (95 pages)

Abstract: Included in this report, a companion volume to the ATC-21 and ATC-21-1 reports, is state-of-the-art information on (1) the identification of those buildings that might collapse and trap victims in debris or generate debris of such a size that its handling would require special or heavy lifting equipment; (2) guidance in identifying these types of buildings, on the basis of their major exterior features, and (3) the types and life capacities of equipment required to remove the heavy portion of the debris that might result from the collapse of such buildings.

ATC-22: The report, *A Handbook for Seismic Evaluation of Existing Buildings (Preliminary)*, was developed under a contract from FEMA. Available through the ATC office. (169 pages)

Abstract: This handbook provides methodology for seismic evaluation of existing buildings of different types and occupancies in areas of different seismicity throughout the United States. The methodology, which has been field tested in several programs nationwide, utilizes the information and procedures developed for and documented in the ATC-14 report. The handbook includes checklists, diagrams, and sketches designed to assist the user.

ATC-22-1: The report, *Seismic Evaluation of Existing Buildings: Supporting Documentation*, was developed under a contract from FEMA. Available through the ATC office. (160 pages)

Abstract: Included in this report, a companion volume to the ATC-22 report, are (1) a review and evaluation
of existing buildings seismic evaluation methodologies; (2) results from field tests of the ATC-14 methodology; and (3) summaries of evaluations of ATC-14 conducted by the National Center for Earthquake Engineering Research (State University of New York at Buffalo) and the City of San Francisco.

ATC-23A: The report, General Acute Care Hospital Earthquake Survivability Inventory for California, Part A: Survey Description, Summary of Results, Data Analysis and Interpretation, was developed under a contract from the Office of Statewide Health Planning and Development (OSHPD), State of California. Available through the ATC office. (58 pages)

Abstract: This report, completed in 1991, summarizes results from a seismic survey of 490 California acute care hospitals. Included are a description of the survey procedures and data collected, a summary of the data, and an illustrative discussion of data analysis and interpretation that has been provided to demonstrate potential applications of the ATC-23 database.

ATC-23B: The report, General Acute Care Hospital Earthquake Survivability Inventory for California, Part B: Raw Data, is a companion document to the ATC-23A Report and was developed under the same contract from OSHPD. Available through the ATC office. (377 pages)

Abstract: Included in this report, completed in 1991, are tabulations of raw general site and building data for 490 acute care California hospitals in California.

ATC-24: The report, Guidelines for Seismic Testing of Components of Steel Structures, was jointly funded by the American Iron and Steel Institute (AISI), American Institute of Steel Construction (AISC), National Center for Earthquake Engineering Research (NCEER), and NSF. Available through the ATC office. (57 pages)

Abstract: This report, completed in 1992, provides guidance for most cyclic experiments on components of steel structures for the purpose of consistency in experimental procedures. The report contains recommendations and companion commentary pertaining to loading histories, presentation of test results, and other aspects of experimentation. The recommendations are written specifically for experiments with slow cyclic load application.

ATC-25: The report, Seismic Vulnerability and Impact of Disruption of Lifelines in the Conterminous United States, was developed under a contract from FEMA. Available through the ATC office. (440 pages)

Abstract: Documented in this report is a national overview of lifeline seismic vulnerability and impact of disruption. Lifelines considered include electric systems, water systems, transportation systems, gas and liquid fuel supply systems, and emergency service facilities (hospitals, fire and police stations). Vulnerability estimates and impacts developed are presented in terms of estimated first approximation direct damage losses and indirect economic losses.

ATC-25-1: The report, A Model Methodology for Assessment of Seismic Vulnerability and Impact of Disruption of Water Supply Systems, was developed under a contract from FEMA. Available through the ATC office. (147 pages)

Abstract: This report contains a practical methodology for the detailed assessment of seismic vulnerability and impact of disruption of water supply systems. The methodology has been designed for use by water supply system operators. Application of the methodology enables the user to develop estimates of direct damage to system components and the time required to restore damaged facilities to pre-earthquake usability. Suggested measures for mitigation of seismic hazards are also provided.
ATC-29: The report, *Proceedings of Seminar and Workshop on Seismic Design and Performance of Equipment and Nonstructural Elements in Buildings and Industrial Structures*, was developed under a grant from NCEER and NSF. Available through the ATC office. (470 pages)

Abstract: These Proceedings contain 35 papers describing state-of-the-art technical information pertaining to the seismic design and performance of equipment and nonstructural elements in buildings and industrial structures. Included are papers describing current practice, codes and regulations; earthquake performance; analytical and experimental investigations; development of new seismic qualification methods; and research, practice, and code development needs for specific elements and systems. The report also includes a summary of a proposed 5-year research agenda for NCEER.

ATC-30: The report, *Proceedings of Workshop for Utilization of Research on Engineering and Socioeconomic Aspects of 1985 Chile and Mexico Earthquakes*, was developed under a grant from NSF. Available through the ATC office. (113 pages)

Abstract: This report documents the findings of a 1990 technology transfer workshop in San Diego, California, co-sponsored by ATC and the Earthquake Engineering Research Institute. Included in the report are invited papers and working group recommendations on geotechnical issues, structural response issues, architectural and urban design considerations, emergency response planning, search and rescue, and reconstruction policy issues.

ATC-31: The report, *Evaluation of the Performance of Seismically Retrofitted Buildings*, was developed under a contract from the National Institute of Standards and Technology and funded by the U. S. Geological Survey. Available through the ATC office. (75 pages)

Abstract: This report summarizes the results from an investigation of the effectiveness of 229 seismically retrofitted buildings, primarily unreinforced masonry and concrete tilt-up buildings. All buildings were located in the areas affected by the 1987 Whittier Narrows, California, and 1989 Loma Prieta, California, earthquakes.
### ATC BOARD OF DIRECTORS (1973-Present)

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<td>Albert J. Blaylock</td>
<td>(1976-77)</td>
<td>Stephen McReavy</td>
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<td>Anil Chopra</td>
<td>(1973-74)</td>
<td>Gary Morrison</td>
<td>(1973)</td>
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<td>Richard Christopherson</td>
<td>(1976-80)</td>
<td>Robert Morrison</td>
<td>(1981-84)</td>
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<td>Burke A. Draheim</td>
<td>(1973-74)</td>
<td>Sherrill Pitkin</td>
<td>(1984-87)</td>
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<td>John E. Droeger</td>
<td>(1973)</td>
<td>Edward V. Podlack</td>
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<td>(1986-89)</td>
<td>Lawrence D. Reaveley*</td>
<td>(1985-91)</td>
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<td>William J. Hall</td>
<td>(1985-86)</td>
<td>C. Mark Saunders</td>
<td>(1993-96)</td>
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<td>Donald R. Strand</td>
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<td>Robert S. White</td>
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<td>Walter B. Lum</td>
<td>(1975-78)</td>
<td>* President</td>
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<td>Kenneth A. Luttrel</td>
<td>(1991-94)</td>
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<td>Melvyn H. Mark</td>
<td>(1979-82)</td>
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### ATC EXECUTIVE DIRECTORS (1973-Present)

<table>
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<tr>
<th>Name</th>
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<tr>
<td>Christopher Rojahn</td>
<td>(1981-present)</td>
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