Chapter 15 Commentary

STRUCTURES WITH DAMPING SYSTEMS

Background. Chapter 15, Structures with Damping Systems, appears for the first time in the body of to the 2003 *Provisions*, having first appeared as an appendix (to Chapter 13) in the 2000 *Provisions*. The appendix was developed by Technical Subcommittee 12 (TS 12) of the Provision Update Committee (PUC) during the 2000 update cycle to provide a basis for designing structures with damping systems that is consistent with the *NEHRP Provisions*, in particular structures with seismic (base) isolation systems. Voting members of TS 12 during the 2000 update were Dr. Charles Kircher (TS 12 Chair and PUC representative), Dr. Michael Constantinou (PUC representative), Dr. Ian Aiken, Dr. Robert Hanson, Mr. Martin Johnson, Dr. Andrew Taylor, and Dr. Andrew Whittaker

During the 2000 update cycle, the primary resource documents for the design of structures with dampers were the *NEHRP Guidelines for Seismic Rehabilitation of Buildings* (FEMA 273, 1997) and the *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 274, 1997). While suitable for the performance-based design, terms, methods of analysis and response limits of the *NEHRP Guidelines* for existing buildings do not match those of the *NEHRP Provisions* for new structures. Accordingly, TS-12 developed new provisions, in particular new linear analysis methods, for design of structures with dampers.

New analysis methods were developed for structures with dampers based on nonlinear “pushover” characterization of the structure and calculation of peak response using effective (secant) stiffness and effective damping properties of the first (pushover) mode in the direction of interest. These are same concepts used in Chapter 13 to characterize the force-deflection properties of isolation systems, modified to explicitly incorporate the effects of ductility demand (post-yield response) and higher-mode response of structures with dampers. In contrast to isolated structures, structures with dampers are in general expected to yield during strong ground shaking (similar to conventional structures), and their performance can be significantly influenced by response of higher modes.

During the 2000 cycle, analysis methods were evaluated using design examples. Response calculated using linear analysis was found to compare well with the results of nonlinear time history analysis (Ramirez, 2001). Additional design examples illustrating explicit “pushover” modeling of the structure may be found in Chapter 9 commentary of FEMA 274. The reader is also referred to Ramirez et al. (2002a, 2002b, 2003) and Whittaker et al. (2003) for a detailed exposition of the analysis procedures in this chapter, background research studies, examples of application and an evaluation of accuracy of the linear static and response spectrum analysis methods.

The balance of this section provides background on the underlying philosophy used by TS12 to develop the chapter, the definition the damping system, the concept of effective damping, and the calculation of earthquake response using either linear or nonlinear analysis methods.

Design Philosophy. The basic approach taken by TS12 in developing the chapter for structures with damping systems is based on the following concepts:

1. The chapter is applicable to all types of damping systems, including both displacement-dependent damping devices of hysteretic or friction systems and velocity-dependent damping devices of viscous or visco elastic systems (Constantinou et al. 1998, Hanson and Soong, 2001)
2. The chapter provides minimum design criteria with performance objectives comparable to those for a structure with a conventional seismic-force-resisting system (but also permits design criteria that will achieve higher performance levels).

3. The chapter requires structures with a damping system to have a seismic-force-resisting system that provides a complete load path. The seismic-force-resisting system must comply with the requirements of the *Provisions*, except that the damping system may be used to meet drift limits.
4. The chapter requires design of damping devices and prototype testing of damper units for displacements, velocities, and forces corresponding to those of the maximum considered earthquake (same approach as that used for structures with an isolation system).
5. The chapter provides linear static and response spectrum analysis methods for design of most structures that meet certain configuration and other limiting criteria (for example, at least two damping devices at each story configured to resist torsion). The chapter requires additional nonlinear response history analysis to confirm peak response for structures not meeting the criteria for linear analysis (and for structures close to major faults).

Damping system. The chapter defines the damping system as:

> "The collection of structural elements that includes all individual damping devices, all structural elements or bracing required to transfer forces from damping devices to the base of the structure, and all structural elements required to transfer forces from damping devices to the seismic-force-resisting system."

The damping system is defined separately from the seismic-force-resisting system, although the two systems may have common elements. As illustrated in Figure C15-1, the damping system may be external or internal to the structure and may have no shared elements, some shared elements, or all elements in common with the seismic-force-resisting system. Elements common to the damping system and the seismic-force-resisting system must be designed for a combination of the two loads of the two systems.

The seismic-force-resisting system may be thought of as a collection of lateral-force-resisting elements of the structure if the damping system was not functional (as if damping devices were disconnected). This system is required to be designed for not less than 75 percent of the base shear of a conventional structure (not less than 100 percent, if the structure is highly irregular), using an R factor as defined in Table 4.3-1. This system provides both a safety net against damping system malfunction as well as the stiffness and strength necessary for the balanced lateral displacement of the damped structure.

The chapter requires the damping system to be designed for the actual (non-reduced) earthquake forces (such as, peak force occurring in damping devices). For certain elements of the damping system, other than damping devices, limited yielding is permitted provided such behavior does not affect damping system function or exceed the amount permitted by the *Provisions* for elements of conventional structures.

The chapter defines a damping device as:

> "A flexible structural element of the damping system that dissipates energy due to relative motion of each end of the device. Damping devices include all pins, bolts, gusset plates, brace extensions, and other components required to connect damping devices to other elements of the structure. Damping devices may be classified as either displacement-dependent or velocity-dependent, or a combination thereof, and may be configured to act in either a linear or nonlinear manner."

Following the same approach as that used for design of seismic isolators, damping devices must be designed for maximum considered earthquake displacements, velocities, and forces. Likewise, prototype damper units must be fully tested to demonstrate adequacy for maximum considered earthquake loads and to establish design properties (such as effective damping).

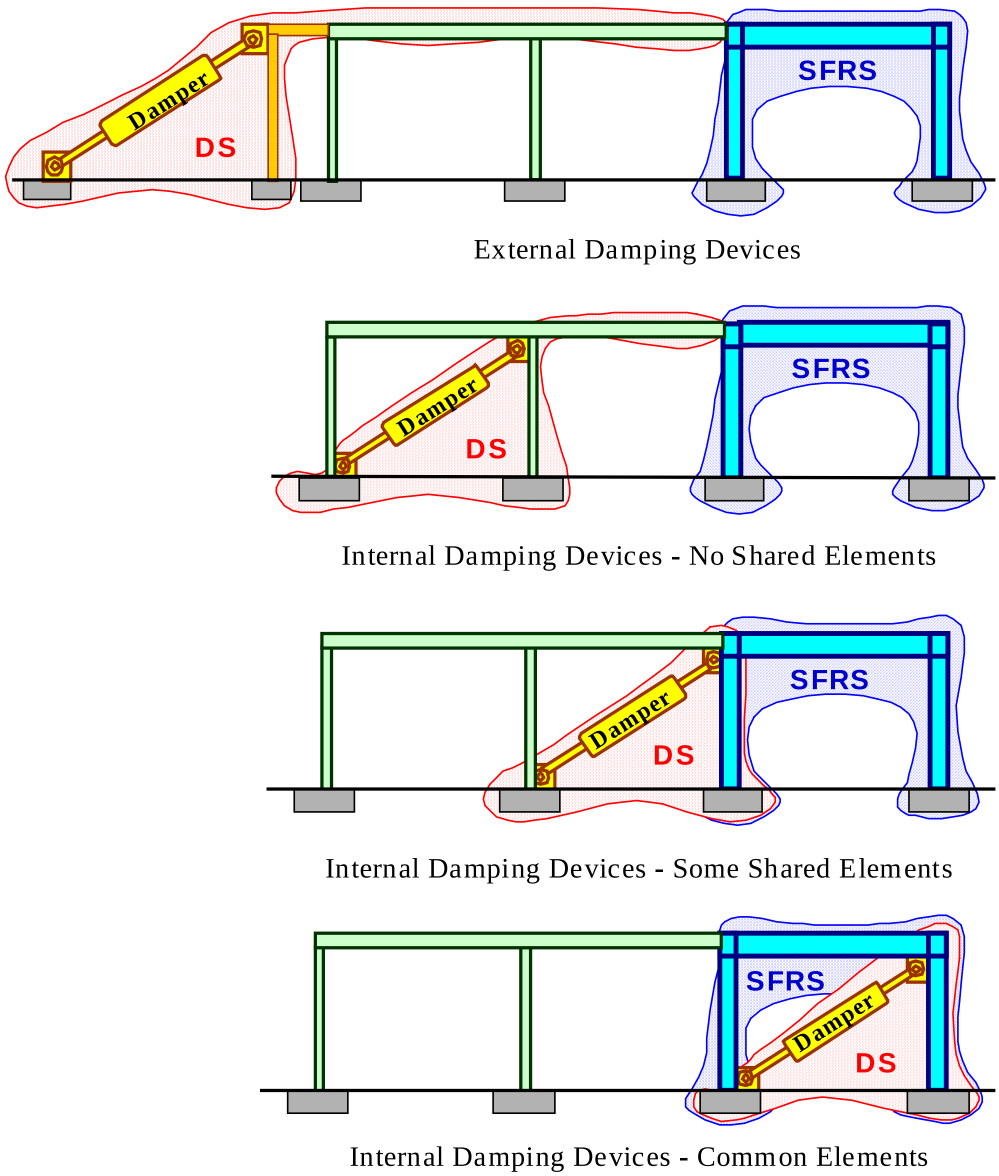

Figure C15-1. Damping system (DS) and seismic-force-resisting system (SFRS) configurations.

Effective Damping. The chapter reduces the response of a structure with a damping system by the damping coefficient, B, based on the effective damping, β, of the mode of interest. This is the same approach as that used by the *Provisions* for isolated structures. Values of the B coefficient recommended for design of damped structures are the same as those in the *Provisions* for isolated structures at damping levels up to 30 percent, but now extend to higher damping levels based on the results presented in Ramirez et al. (2001). Like isolation, effective damping of the fundamental-mode of a damped structure is based on the nonlinear force-deflection properties of the structure. For use with linear analysis methods, nonlinear properties of the structure are inferred from overstrength, Ω_0, and other terms of the *Provisions*. For nonlinear analysis methods, properties of the structure would be based on explicit modeling of the post-yield behavior of elements.

Figure C15-2 illustrates reduction in design earthquake response of the fundamental mode due to effective damping coefficient, B_{1D}. The capacity curve is a plot of the nonlinear behavior of the fundamental mode in spectral acceleration/displacement coordinates. Damping reduction is applied at the effective period of the fundamental mode of vibration (based on the secant stiffness).

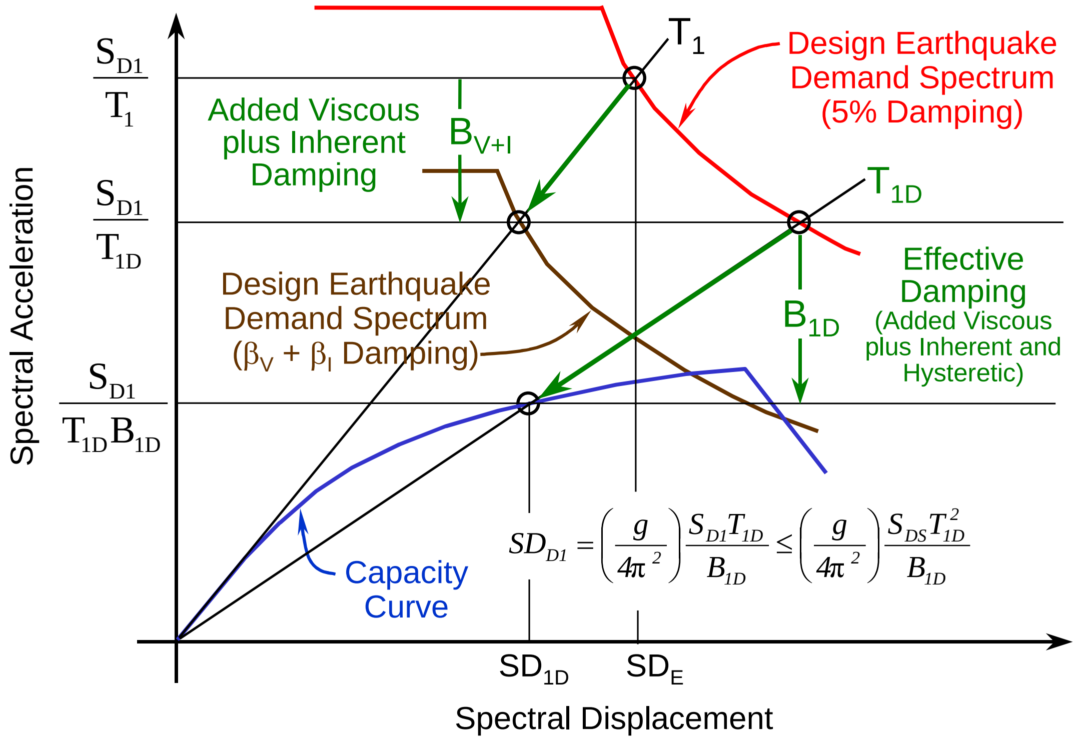

Figure C15-2. Effective damping reduction of design demand.

In general, effective damping is a combination of three components:

1. Inherent Damping β_I—Inherent damping of structure at or just below yield, excluding added viscous damping (typically assumed to be 5 percent of critical for structural systems without dampers).
2. Hysteretic Damping β_H—Post-yield hysteretic damping of the seismic-force-resisting system at the amplitude of interest (taken as 0 percent of critical at or below yield).
3. Added Viscous Damping β_V—Viscous component of the damping system (taken as 0 percent for hysteretic or friction-based damping systems).

Both hysteretic damping and the effects of added viscous damping are amplitude-dependent and the relative contributions to total effective damping changes with the amount of post-yield response of the structure. For example, adding dampers to a structure decreases post-yield displacement of the structure and hence decreases the amount of hysteretic damping provided by the seismic-force-resisting system. If the displacements were reduced to the point of yield, the hysteretic component of effective damping would be zero and the effective damping would be equal to inherent damping plus added viscous damping. If there were no damping system (as in a conventional structure), then effective damping would simply be equal to inherent damping (typically assumed to be 5 percent of critical for most conventional structures).

Linear Analysis Methods. The chapter specifies design earthquake displacements, velocities, and forces in terms of design earthquake spectral acceleration and modal properties. For equivalent lateral force (ELF) analysis, response is defined by two modes: (1) the fundamental mode, and **(2) the residual**

mode. The residual mode is a new concept used to approximate the combined effects of higher modes. While typically of secondary importance to story drift, higher modes can be a significant contributor to story velocity and hence are important for design of velocity-dependent damping devices. For response spectrum analysis, higher modes are explicitly evaluated.

For both the ELF and the response spectrum analysis procedures, response in the fundamental mode in the direction of interest is based on assumed nonlinear (pushover) properties of the structure. Nonlinear (pushover) properties, expressed in terms of base shear and roof displacement, are related to building capacity, expressed in terms of spectral coordinates, using mass participation and other fundamental-mode factors shown in Figure C15-3. The conversion concepts and factors shown in Figure C15-3 are the same as those defined in Chapter 9 of *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 273), which addresses seismic rehabilitation of a structure with damping devices.

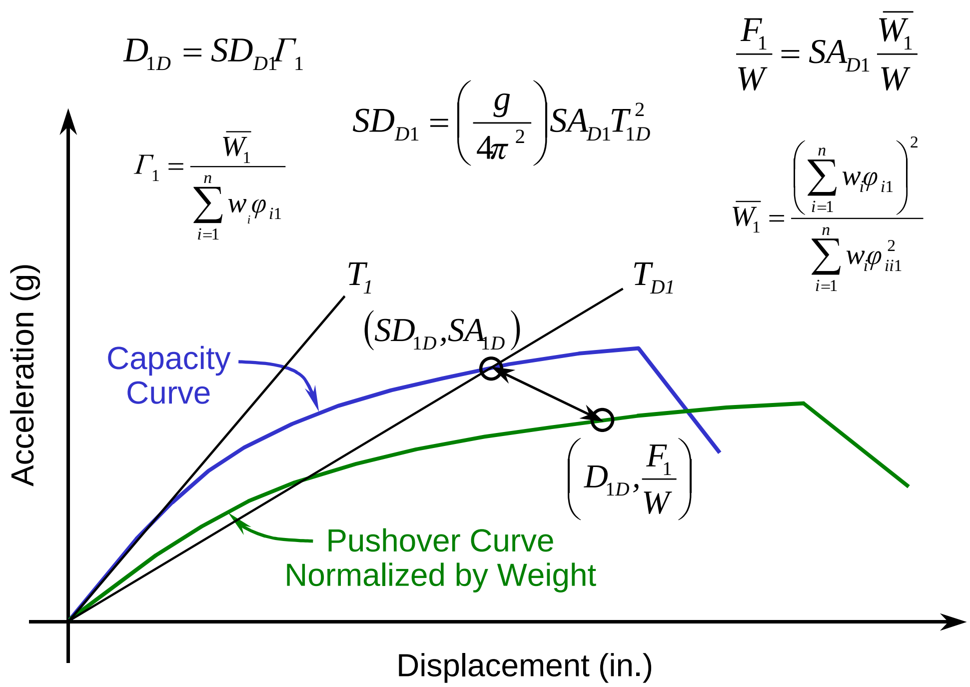

Figure C15-3. Pushover and capacity curves.

When using linear analysis methods, the shape of the fundamental-mode pushover curve is not known and an idealized elasto-plastic shape is assumed, as shown in Figure C15-4. The idealized pushover curve shares a common point with the actual pushover curve at the design earthquake displacement, D_{1D}. The idealized curve permits defining global ductility demand due to the design earthquake, μ_D, as the ratio of design displacement, D_{1D}, to the yield displacement, D_Y. This ductility factor is used to calculate various design factors and to set limits on the building ductility demand, μ_{max}, which limits are consistent with conventional building response limits. Design examples using linear analysis methods have been developed and found to compare well with the results of nonlinear time history analysis (Ramirez et al., 2001).

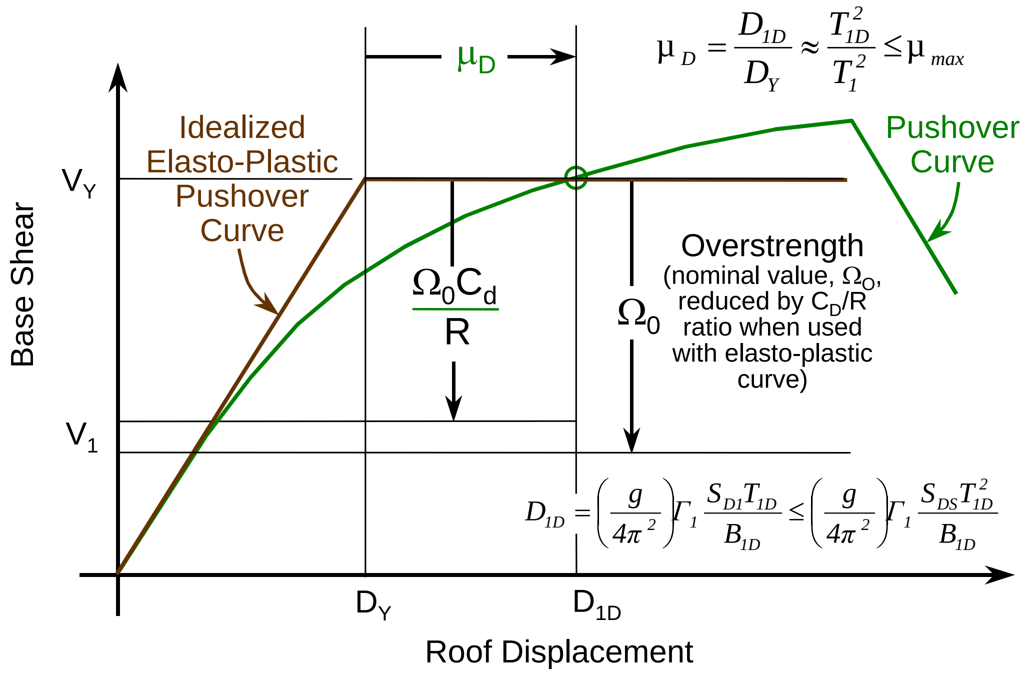

Figure C15-4. Idealized elasto-plastic pushover curve used for linear analysis.

The chapter requires elements of the *damping system* to be designed for actual fundamental-mode design earthquake forces corresponding to a base shear value of V_Y (except that damping devices are designed and prototypes tested for maximum considered earthquake response). Elements of the seismic-force-resisting system are designed for reduced fundamental-mode base shear, V_1, where force reduction is based on system overstrength, Ω_0, conservatively decreased by the ratio, C_d/R, for elastic analysis (when actual pushover strength is not known).

Nonlinear analysis methods. The chapter specifies procedures for the nonlinear response history analyses and a nonlinear static procedure. For designs in which the seismic-force-resisting-system will remain elastic, only the nonlinear damping device characteristics need to be modeled for these analyses.

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Commentary Appendix A

DEVELOPMENT OF MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION MAPS FIGURES 3.3-1 THROUGH 3.3-14

BACKGROUND

The maps used in the *Provisions* through 1994 provided the A_a (effective peak acceleration coefficient) and A_v (effective peak velocity-related acceleration coefficient) values to use for design. The BSSC had always recognized that the maps and coefficients would change with time as the profession gained more knowledge about earthquakes and their resulting ground motions and as society gained greater insight into the process of establishing acceptable risk.

By 1997, significant additional earthquake data had been obtained that made the A_a and A_v maps, then about 20 years old, seriously out of date. For the 1997 *Provisions*, a joint effort involving the BSSC, the Federal Emergency Management Agency (FEMA), and the U.S. Geological Survey (USGS) was conducted to develop both new maps for use in design and new design procedures reflecting the significant advances made in the past 20 years. The BSSC's role in this joint effort was to develop new ground motion maps for use in design and design procedures based on new USGS seismic hazard maps.

The BSSC appointed a 15-member Seismic Design Procedure Group (SDPG) to develop the seismic ground motion maps and design procedures. The SDPG membership was composed of representatives of different segments of the design community as well as two earth science members designated by the USGS, and the membership was representative of the different geographical regions of the country. Also, the BSSC, with input from FEMA and USGS, appointed a five-member Management Committee (MC) to guide the efforts of the SDPG. The MC was geographically balanced insofar as practicable and was composed of two seismic hazard definition experts and three engineering design experts, including the chairman of the 1997 *Provisions* Update Committee (PUC). The SDPG and the MC worked closely with the USGS to define the BSSC mapping needs and to understand how the USGS seismic hazard maps should be used to develop the BSSC seismic ground motion maps and design procedures.

For a brief overview of how the USGS developed its hazard maps, see Appendix B to this *Commentary* volume. A detailed description of the development of the maps is contained in the USGS Open-File Report 96-532, *National Seismic-Hazard Maps: Documentation, June 1996*, by Frankel, et al. (1996). The USGS hazard maps also can be viewed and printed from a USGS Internet site at http://eqhazmaps.usgs.gov.

The goals of the SDPG were as follows:

1. To replace the existing effective peak acceleration and velocity-related acceleration design maps with new ground motion spectral response maps based on new USGS seismic hazard maps.
2. To develop the new ground motion spectral response maps within the existing framework of the *Provisions* with emphasis on uniform margin against the collapse of structures.
3. To develop design procedures for use with the new ground motion spectral response maps.

PURPOSE OF THE *PROVISIONS*

The purpose of the *Provisions* is to present criteria for the design and construction of new structures subject to earthquake ground motions in order to minimize the risk to life for all structures, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential structures to function after an earthquake. To this end, the *Provisions* provide the minimum criteria considered prudent for structures subjected to earthquakes at any location in the United States and its territories. The *Provisions* generally considers property damage as it relates to occupant safety for ordinary structures. For high occupancy and essential structures, damage limitation criteria are more strict in order to better provide for the safety of occupants and the continued functioning of the structure. Some structural and nonstructural damage can be expected as a result of the "design ground motions" because the *Provisions* allow inelastic energy dissipation by utilizing the deformability of the structural system. For ground motions in excess of the design levels, the intent is that there be a low likelihood of collapse. These goals of the *Provisions* were the guiding principles for developing the design maps.

POLICY DECISIONS FOR SEISMIC GROUND MOTION MAPS

The new maps (cited in both the 1997 and 2000 *Provisions*) reflect the following policy decisions that depart from past practice and the 1994 *Provisions*:

1. The maps define the maximum considered earthquake ground motion for use in design procedures,
2. The use of the maps for design provide an approximately uniform margin against collapse for ground motions in excess of the design levels in all areas.
3. The maps are based on both probabilistic and deterministic seismic hazard maps, and
4. The maps are response spectra ordinate maps and reflect the differences in the short-period range of the response spectra for the areas of the United States and its territories with different ground motion attenuation characteristics and different recurrence times.

These policy decisions reflected new information from both the seismic hazard and seismic engineering communities that is discussed below.

In the 1994 *Provisions*, the design ground motions were based on an estimated 90 percent probability of not being exceeded in 50 years (about a 500 year mean recurrence interval) (ATC 3-06 1978). The 1994 *Provisions* also recognized that larger ground motions are possible and that the larger motions, although their probability of occurrence during a structure's life is very small, nevertheless can occur at any time. The 1994 *Provisions* also defined a maximum capable earthquake as "the maximum level of earthquake ground shaking that may ever be expected at the building site within the known geologic framework." It was additionally specified that in certain map areas ($\geq A_a = 0.3$), the maximum capable earthquake was associated with a motion that has a 90 percent probability of not being exceeded in 100 years (about a 1000 year mean recurrence interval). In addition to the maximum capable earthquake definition, sample ground motion maps were prepared with 90 percent probabilities of not being exceeded in 250 years (about a 2500 year mean recurrence interval).

Given the wide range in return periods for maximum magnitude earthquakes throughout the United States and its territories (100 years in parts of California to 100,000 years or more in several other locations), current efforts have focused on defining the maximum considered earthquake ground motions for use in design (not the same as the maximum capable earthquake defined in the 1994 *Provisions*). The maximum considered earthquake ground motions are determined in a somewhat different manner depending on the seismicity of an individual region; however, they are uniformly defined as the maximum level of earthquake ground shaking that is considered as reasonable to design structures to resist. Focusing on ground motion versus earthquake size facilitates the development of a design approach that provides an approximately uniform margin against collapse throughout the United States.

As noted above, the 1994 *Provisions* generally used the notation of 90 percent probability of not being exceeded in a certain exposure time period (50, 100, or 250 years), which can then be used to calculate a given mean recurrence interval (500, 1000, or 2500 years). For the purpose of the new maps and design procedure introduced in the 1997 *Provisions*, the single exposure time period of 50 years has been commonly used as a reference period over which to consider loads on structures (after 50 years of use, structures may require evaluation to determine future use and rehabilitation needs). With this in mind, different levels of probability or return period are expressed as percent probability of exceedance in 50 years. Specifically, 10 percent probability of exceedance in 50 years is a mean recurrence interval of about 500 years, 5 percent probability of exceedance in 50 years is a mean recurrence interval of about 1000 years, and 2 percent probability of exceedance in 50 years is a mean recurrence interval of about 2500 years. The above notation is used throughout the *Provisions*.

Review of modern probabilistic seismic hazard results, including the maps prepared by the USGS to support the effort resulting in the 1997 *Provisions*, indicates that the rate of change of ground motion versus probability is not constant throughout the United States. For example, the ground motion difference between the 10 percent probability of exceedance and 2 percent probability of exceedance in 50 years in coastal California is typically smaller than the difference between the two probabilities in less active seismic areas such as the eastern or central United States. Because of these differences, questions were raised concerning whether definition of the ground motion based on a constant probability for the entire United States would result in similar levels of seismic safety for all structures. Figure A1 plots the 0.2 second spectral acceleration normalized at 2 percent probability of exceedance in 50 years versus the annual frequency of exceedance. Figure A1 shows that in coastal California, the ratio between the 0.2 second spectral acceleration for the 2 and the 10 percent probabilities of exceedance in 50 years is about 1.5 whereas, in other parts of the United States, the ratio varies from 2.0 to 5.0.

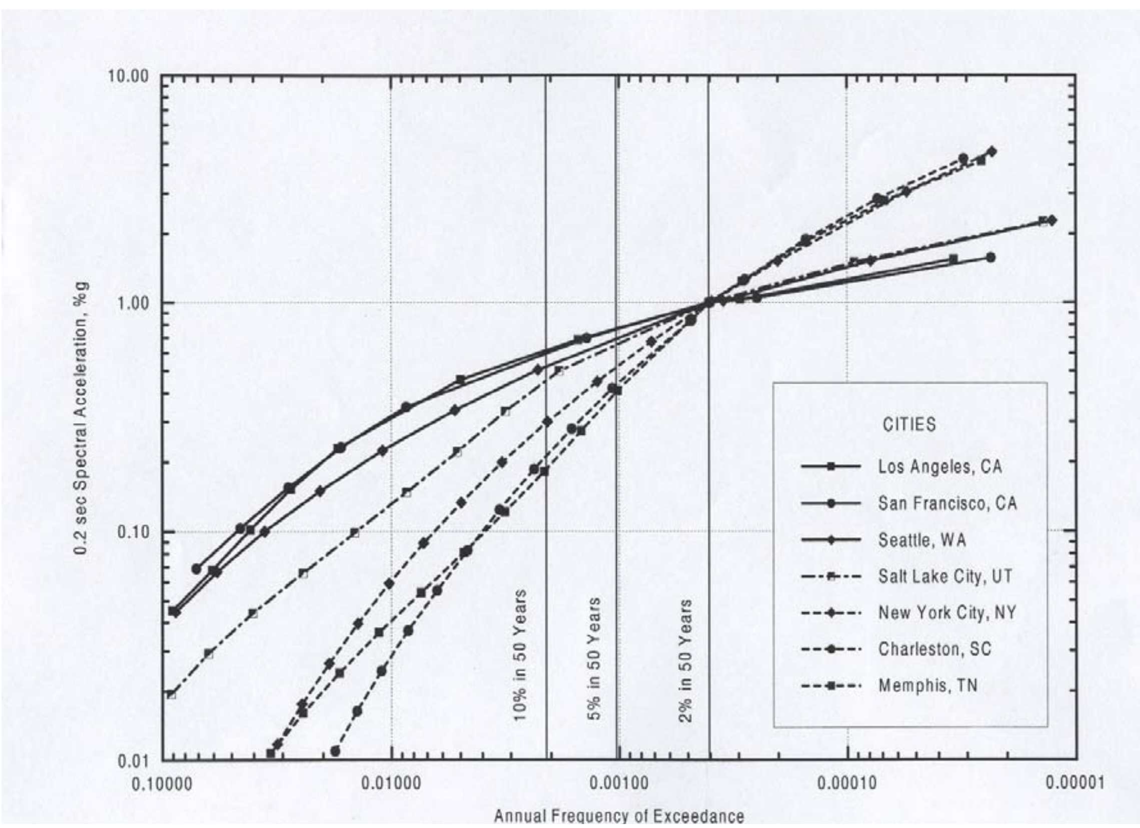

FIGURE A1 Relative hazard at selected sites for 0.2 sec spectral response acceleration. The hazard curves are normalized at 2 percent probability of exceedance in 50 years.

In answering the questions, it was recognized that seismic safety is the result of a number of steps in addition to defining the design earthquake ground motions, including the critical items generally defined as proper site selection, structural design criteria, analysis and procedures, detailed design requirements, and construction. The conservatism in the actual design of the structure is often referred to as the "seismic margin." It is the seismic margin that provides confidence that significant loss of life will not be caused by actual ground motions equal to the design levels. Alternatively, the seismic margin provides a level of protection against larger, less probable earthquakes although at a lower level of confidence.

The collective opinion of the SDPG was that the seismic margin contained in the *Provisions* provides, as a minimum, a margin of about 1.5 times the design earthquake ground motions. In other words, if a structure experiences a level of ground motion 1.5 times the design level, the structure should have a low likelihood of collapse. The SDPG recognizes that quantification of this margin is dependent on the type of structure, detailing requirements, etc., but the 1.5 factor is a conservative judgment appropriate for structures designed in accordance with the *Provisions*. This seismic margin estimate is supported by Kennedy et al. (1994), Cornell (1994), and Ellingwood (1994) who evaluated structural design margins and reached similar conclusions.

The USGS seismic hazard maps indicate that in most locations in the United States the 2 percent probability of exceedance in 50 years ground motion values are more than 1.5 times the 10 percent probability of exceedance in 50 years ground motion values. This means that if the 10 percent probability of exceedance in 50 years map was used as the design map and the 2 percent probability of exceedance in 50 years ground motions were to occur, there would be low confidence (particularly in the central and eastern United States) that structures would not collapse due to these larger ground motions. Such a conclusion for most of the United States was not acceptable to the SDPG. The only location where the above results seemed to be acceptable was coastal California (2 percent probability of exceedance in 50 years map is about 1.5 times the 10 percent probability of exceedance in 50 years map) where structures have experienced levels of ground shaking equal to and above the design value.

The USGS probabilistic seismic hazard maps for coastal California also indicate the 10 percent probability of exceedance in 50 years seismic hazard map is significantly different from (in most cases larger) the design ground motion values contained in the 1994 *Provisions*. Given the generally successful experience with structures that complied with the recent editions of the *Uniform Building Code* whose design map contained many similarities to the 1994 *Provisions* design map, the SDPG was reluctant to suggest large changes without first understanding the basis for the changes. This stimulated a detailed review of the probabilistic maps for coastal California. This review identified a unique issue for coastal California in that the recurrence interval of the estimated maximum magnitude earthquake is less than the recurrence interval represented on the probabilistic map, in this case the 10 percent probability of exceedance in 50 years map (i.e., recurrence interval for maximum magnitude earthquake is 100 to 200 years versus 500 years.)

Given the above, one choice was to accept the change and use the 10 percent probability of exceedance in 50 years probabilistic map to define the design ground motion for coastal California and, using this, determine the appropriate probability for design ground motion for the rest of the United States that would result in the same level of seismic safety. This would have resulted in the design earthquake being defined at 2 percent probability of exceedance in 50 years and the need for development of a 0.5 to 1.0 percent probability of exceedance in 50 years map to show the potential for larger ground motions outside of coastal California. Two major problems were identified. The first is that requiring such a radical change in design ground motion in coastal California seems to contradict the general conclusion that the seismic design codes and process are providing an adequate level of life safety. The second is that completing probabilistic estimates of ground motion for lower probabilities (approaching those used for critical facilities such as nuclear power plants) is associated with large uncertainties and can be quite controversial.

An alternative choice was to build on the observation that the maximum earthquake for many seismic faults in coastal California is fairly well known and associated with probabilities larger than a 10 percent probability of exceedance in 50 years (500 year mean recurrence interval). Given this, a decision was made to develop a procedure that would use the best estimate of ground motion from maximum magnitude earthquakes on seismic faults with high probabilities of occurrence (short return periods). For the purposes of the *Provisions,* these earthquakes are defined as "deterministic earthquakes." Following this approach and recognizing the inherent seismic margin contained in the *Provisions,* it was determined that the level of seismic safety achieved in coastal California would be approximately equivalent to that associated with a 2 to 5 percent probability of exceedance in 50 years for areas outside of coastal California. In other words, the use of the deterministic earthquakes to establish the maximum considered earthquake ground motions for use in design in coastal California results in a level of protection close to that implied in the 1994 *Provisions* and consistent with maximum magnitude earthquakes expected for those seismic sources. Additionally, this approach results in less drastic changes to ground motion values for coastal California than the alternative approach of using probabilistic based maps.

One could ask why any changes are necessary for coastal California given the positive experience from recent earthquakes. While it is true that the current seismic design practices have produced positive results, the current design ground motions in the 1994 *Provisions* are less than those expected from maximum magnitude earthquakes on known seismic sources. The 1994 *Provisions* reportedly considered maximum magnitude earthquakes but did not directly link them to the design ground motions (Applied Technology Council, 1978). If there is high confidence in the definition of the fault and magnitude of the earthquake and the maximum earthquake occurs frequently, then the design should be linked to at least the best estimate ground motion for such an earthquake. Indeed, it is the actual earthquake experience in coastal California that is providing increased confidence in the seismic margins contained in the *Provisions.*

The above approach also is responsive to comments that the use of 10 percent probability of exceedance in 50 years is not sufficiently conservative in the central and eastern United States where the earthquakes are expected to occur infrequently. Based on the above discussion and the inherent seismic margin contained in the *Provisions*, the SDPG selected 2 percent probability of exceedance in 50 years as the maximum considered earthquake ground motion for use in design where the use of the deterministic earthquake approach discussed above is not used.

The maximum considered earthquake ground motion maps are based on two response spectral values (a short-period and a long-period value) instead of the A_a and A_v coefficients. The decision to use response spectral values is based on earthquake data obtained during the past 20 years showing that site-specific spectral values are more appropriate for design input than the A_a and A_v coefficients used with standardized spectral shapes. The spectral shapes vary in different areas of the country and for different site conditions. This is particularly the case for the short-period portion of the response spectra. Based on the differences in the ground motion attenuation characteristics between the central and eastern and western United States, the USGS used different ground motion attenuation functions for these areas in developing the seismic hazard maps. The ground motion attenuation functions in the eastern United States result in higher short-period spectral accelerations at lower periods for a given earthquake magnitude than the western United States attenuation functions, particularly compared to the high seismicity region of coastal California. The short-period response spectral values were reviewed in order to determine the most appropriate value to use for the maximum considered earthquake ground motion maps. Based on this review, the short-period spectral response value of 0.2 second was selected to represent the short-period range of the response spectra for the eastern United States. In the western United States the most appropriate short-period response spectral value was determined to be 0.3 second, but a comparison of the 0.2 and 0.3 second values indicated that the differences in the response spectral values were insignificant. Based on this and for convenience of preparing the maximum considered earthquake ground motion maps, the short-period response spectral value of 0.2 second was selected to represent the short-period range of the response spectra for all of the United States. The long-period response spectral value selected for use is 1.0 second for all of the United States. Based on the ground motion attenuation functions and the USGS seismic hazard maps, a $1/T$ (T = natural period) relationship was selected to define the response spectra from the short period value to the long-period value. Using the spectral values from the ground motion maps will allow the different spectral shapes to be incorporated into design.

DEVELOPMENT OF THE MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION MAPS FOR USE IN DESIGN

The concept for developing maximum considered earthquake ground motions for use in design involved two distinct steps:

1. The various USGS probabilistic seismic hazard maps were combined with deterministic hazard maps by a set of rules (logic) to create the maximum considered earthquake ground motion maps that can be used to define response spectra for use in design and
2. Design procedures were developed that transform the response spectra into design values (e.g., design base shear).

The response spectra defined from the first step represent general "site-dependent" spectra similar to those that would be obtained by a geotechnical study and used for dynamic analysis except their shapes are less refined (i.e., shape defined for only a limited number of response periods). The response spectra do not represent the same hazard level across the country but do represent actual ground motion consistent with providing approximately uniform protection against the collapse of structures. The response spectra represent the maximum considered earthquake ground motions for use in design for Site Class B (rock with a shear wave velocity of 760 meters/second).

The maximum considered earthquake ground motion maps for use in design are based on a defined set of rules for combining the USGS seismic hazard maps to reflect the differences in the ability to define the fault sources and seismicity characteristics across the regions of the country as discussed in the policy decisions. Accommodating regional differences allows the maximum considered earthquake maps to represent ground motions for use in design that provide reasonably consistent margins of preventing the collapse of structures. Based on this, three regions have been defined:

1. Regions of negligible seismicity with very low probability of collapse of the structure,
2. Regions of low and moderate to high seismicity, and
3. Regions of high seismicity near known fault sources with short return periods.

Regions of Negligible Seismicity With Very Low Probability of Collapse of the Structure

The regions of negligible seismicity with very low probability of collapse have been defined by:

1. Determining areas where the seismic hazard is controlled by earthquakes with M_b (body wave magnitude) magnitudes less than or equal to 5.5 and
2. Examining the recorded ground motions associated with Modified Mercalli Intensity V.

The basis for the first premise is that in this region, there are a number of examples of earthquakes with $M_b \cong 5.5$ which caused only localized damage to structures not designed for earthquakes. The basis for the second premise is that Modified Mecalli Intensity V ground motions typically do not cause structural damage. By definition, Modified Mercalli Intensity V ground shaking is felt by most people, displaces or upsets small objects, etc., but typically causes no, or only minor, structural damage in buildings of any type. Modified Mercalli Intensity VI ground shaking is felt by everyone, small objects fall off shelves, etc., and minor or moderate structural damage occurs to weak plaster and masonry construction. Life-threatening damage or collapse of *structures* would not be expected for either Modified Mercalli Intensities V or VI ground shaking. Based on an evaluation of 1994 Northridge earthquake data, regions of different Modified Mercalli Intensity (Dewey, 1995) were correlated with maps of smooth response spectra developed from instrumental recordings

(Sommerville, 1995). The Northridge earthquake provided a sufficient number of instrumental recordings and associated spectra to permit correlating Modified Mercalli Intensity with response spectra. The results of the correlation determined the average response spectrum for each Modified Mercalli Intensity region. For Modified Mercalli Intensity V, the average response spectrum of that region had a spectral response acceleration of slightly greater than 0.25g at 0.3 seconds and a spectral response acceleration of slightly greater than 0.10g at 1.0 seconds. On the basis of these values and the minor nature of damage associated with Modified Mercalli Intensity V, 0.25g (short-period acceleration) and 0.10g (acceleration at a period of 1 second, taken proportional to 1/T) is deemed to be a conservative estimate of the spectrum below which life-threatening damage would not be expected to occur even to the most vulnerable of types of structures. Therefore, this region is defined as areas having maximum considered earthquake ground motions with a 2 percent probability of exceedance in 50 years equal to or less than 0.25g (short period) and 0.10g (long period). The seismic hazard in these areas is generally the result of $M_b \cong 5.5$ earthquakes. In these areas, a minimum lateral force design of 1 percent of the dead load of the structure shall be used in addition to the detailing requirements for the Seismic Design Category A structures.

In these areas it is not considered necessary to specify seismic-resistant design on the basis of a maximum considered earthquake ground motion. The ground motion computed for such areas is determined more by the rarity of the event with respect to the chosen level of probability than by the level of motion that would occur if a small but close earthquake actually did occur. However, it is desirable to provide some protection, both against earthquakes as well as many other types of unanticipated loadings. The requirements for Seismic Design Category A provide a nominal amount of structural integrity that will improve the performance of buildings in the event of a possible, but rare earthquake. The result of design to Seismic Design Category A is that fewer buildings would collapse in the vicinity of such an earthquake.

The integrity is provided by a combination of requirements. First, a complete load path for lateral forces must be identified. Then it must be designed for a lateral force equal to a 1% acceleration on the mass. Lastly, the minimum connection forces specified for Seismic Design Category A must be satisfied.

The 1 percent value has been used in other countries as a minimum value for structural integrity. For many structures, design for the wind loadings specified in the local building codes will normally control the lateral force design when compared to the minimum structural integrity force on the structure. However, many low-rise heavy structures or structures with significant dead loads resulting from heavy equipment may be controlled by the nominal 1 percent acceleration. Also, minimum connection forces may exceed structural forces due to wind in additional structures.

The regions of negligible seismicity will vary depending on the Site Class on which structures are located. The *Provisions* seismic ground motion maps (Maps 1 through 19) are for Site Class B conditions and the region of negligible seismicity for Site Class B is defined where the maximum considered earthquake ground motion short-period values are $\leq$0.25g and the long-period values are $\leq$ 0.10g. The regions of negligible seismicity for the other Site Classes are defined by using the appropriate site coefficients to determine the maximum considered earthquake ground motion for the Site Class and then determining if the short-period values are $\leq$ 0.25g and the long-period values are $\leq$ 0.10g. If so, then the site of the structure is located in the region of negligible seismicity for that Site Class.

Regions of Low and Moderate to High Seismicity

In regions of low and moderate to high seismicity, the earthquake sources generally are not well defined and the maximum magnitude estimates have relatively long return periods. Based on this, probabilistic hazard maps are considered to be the best means to represent the uncertainties and to define the response spectra for these regions. The maximum considered earthquake ground motion for these regions is defined as the ground motion with a 2 percent probability of exceedance in 50 years. The basis for this decision is explained in the policy discussion.

Consideration was given to establishing a separate region of low seismicity and defining a minimum level of ground motion (i.e., deterministic minimum ground motions). This was considered because in the transition between the regions of negligible seismicity to the regions of low seismicity, the ground motions are relatively small and may not be very meaningful for use in seismic design. The minimum level was also considered because the uncertainty in the ground motion levels in the regions of low seismicity is larger than in the regions of moderate to high seismicity. This larger uncertainty may warrant consideration of using higher ground motions (or some minimum level of ground motion) than provided by the maximum considered earthquake ground motions shown on the maps.

The studies discussed above for the regions of negligible seismicity by Dewey (1995) and Sommerville (1995), plus other unpublished studies (to date), were evaluated as a means of determining minimum levels of ground motion for used in design. These studies correlated the Modified Mercalli Intensity data with the recorded ground motions and associated damage. The studies included damage information for a variety of structures which had no specific seismic design and determined the levels of ground motion associated with each Modified Mercalli Intensity. These studies indicate that ground motion levels of about 0.50g short-period spectral response and 0.20g long-period spectral response are representative of Modified Mercalli Intensity VII damage.

Modified Mercalli Intensity VII ground shaking results in negligible damage in buildings of good design and construction, slight to moderate damage in well-built ordinary buildings, considerable damage in poorly-built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), etc. In other words, Modified Mercalli VII ground shaking is about the level of ground motion where significant structural damage may occur and result in life safety concerns for occupants. This tends to suggest that designing structures for ground motion levels below 0.50g short-period spectral response and 0.20g long-period spectral response may not be meaningful.

One interpretation of this information suggests that the ground motion levels for defining the regions of negligible seismicity could be increased. This interpretation would result in much larger regions that require no specific seismic design compared to the 1994 *Provisions*.

Another interpretation of the information suggests establishing a minimum level of ground Motion (at about the Modified Mercalli VII shaking) for regions of low seismicity, in order to transition from the regions of negligible seismicity to the region of moderate to high seismicity. Implementation of a minimum level of ground motion, such as 0.50g for the short-period spectral response and 0.20g for the long-period spectral response, would result in increases (large percentages) in ground motions used for design compared to the 1994 *Provisions*.

Based on the significant changes in past practices resulting from implementing either of the above interpretations, the SDPG decided that additional studies are needed to support these changes. Results of such studies should be considered for future editions of the *Provisions*.

Regions of High Seismicity Near Known Fault Sources With Short Return Periods

In regions of high seismicity near known fault sources with short return periods, deterministic hazard maps are used to define the response spectra maps as discussed above. The maximum considered earthquake ground motions for use in design are determined from the USGS deterministic hazard maps developed using the ground motion attenuation functions based on the median estimate increased by 50 percent. Increasing the median ground motion estimates by 50 percent is deemed to provide an appropriate margin and is similar to some deterministic estimates for a large magnitude characteristic earthquake using ground motion attenuation functions with one standard deviation. Estimated standard deviations for some active fault sources have been determined to be higher than 50 percent, but this increase in the median ground motions was considered reasonable for defining the maximum considered earthquake ground motions for use in design.

Maximum Considered Earthquake Ground Motion Maps for Use in Design

Considering the rules for the three regions discussed above, the maximum considered earthquake ground motion maps for use in design were developed by combining the regions in the following manner:

1. Where the maximum considered earthquake map ground motion values (based on the 2 percent probability of exceedance in 50 years) for Site Class B adjusted for the specific site conditions are $\leq$ 0.25g for the short-period spectral response and $\leq$ 0.10g for the long period spectral response, then the site will be in the region of negligible seismicity and a minimum lateral force design of 1 percent of the dead load of the structure shall be used in addition to the detailing requirements for the Seismic Design Category A structures.
2. Where the maximum considered earthquake ground motion values (based on the 2 percent probability of exceedance in 50 years) for Site Class B adjusted for the specific site conditions are greater than 0.25g for the short-period spectral response and 0.10g for the long-period spectral response, the maximum considered earthquake ground motion values (based on the 2 percent probability of exceedance in 50 years adjusted for the specific site conditions) will be used until the values equal the present (1994 *Provisions*) ceiling design values increased by 50 percent (short period = 1.50g, long period = 0.60g). The present ceiling design values are increased by 50 percent to represent the maximum considered earthquake ground motion values. This will define the sites in regions of low and moderate to high seismicity.
3. To transition from regions of low and moderate to high seismicity to regions of high seismicity with short return periods, the maximum considered earthquake ground motion values based on 2 percent probability of exceedance in 50 years will be used until the values equal the present (1994 *Provisions*) ceiling design values increased by 50 percent (short period = 1.50g, long period = 0.60g). The present ceiling design values are increased by 50 percent to represent maximum considered earthquake ground motion values. When the 1.5 times the ceiling values are reached, then they will be used until the deterministic maximum considered earthquake map values of 1.5g (long period) and 0.60g (short period) are obtained. From there, the deterministic maximum considered earthquake ground motion map values will be used.

In some cases there are regions of high seismicity near known faults with return periods such that the probabilistic map values (2 percent probability of exceedance in 50 years) will exceed the present ceiling values of the 1994 *Provisions* increased by 50 percent and will be less than the deterministic map values. In these regions, the probabilistic map values will be used for the maximum considered earthquake ground motions.

The basis for using present ceiling design values as the transition between the two regions is because earthquake experience has shown that regularly configured, properly designed *structures* performed satisfactorily in past earthquakes. The most significant structural damage experienced in the Northridge and Kobe earthquakes was related to configuration, structural systems, inadequate connection detailing, incompatibility of deformations, and design or construction deficiencies -- not due to deficiency in strength (Structural Engineers Association of California, 1995). The earthquake designs of the structures in the United States (coastal California) which have performed satisfactorily in past earthquakes were based on the criteria in the *Uniform Building Code*. Considering the site conditions of the structures and the criteria in the *Uniform Building Code*, the ceiling design values for these structures were determined to be appropriate for use with the *Provisions* maximum considered earthquake ground motion maps for Site Class B. Based on this, the equivalent maximum considered earthquake ground motion values for the ceiling were determined to be 1.50g for the short period and 0.60g for the long period.

As indicated above there also are some regions of high seismicity near known fault sources with return periods such that the probabilistic map values (2 percent probability of exceedance in 50 years) will exceed the ceiling values of the 1994 *Provisions* increased by 50 percent and also be less than the deterministic map values. In these regions, the probabilistic map values are used for the maximum considered earthquake ground motions.

The near source area in the high seismicity regions is defined as the area where the maximum considered earthquake ground motion values are ≥ 0.75g on the 1.0 second map. In the near source area, *Provisions* Sec. 5.2.3 through 5.2.6 impose additional requirements for certain structures unless the structures are fairly regular, do not exceed 5 stories in height, and do not have a period of vibration over 0.5 seconds. For the fairly regular structures not exceeding 5 stories in height and not having a period of vibration over 0.5 seconds, the maximum considered earthquake ground motion values will not exceed the present ceiling design values increased by 50 percent. The basis for this is because of the earthquake experience discussed above.

These development rules for the maximum considered earthquake ground motion maps for use in design are illustrated in Figures A2 and A3. The application of these rules resulted in the maximum considered earthquake ground motion maps (Maps 1 through 24) introduced in the 1997 and used again in the 2000 *Provisions*.

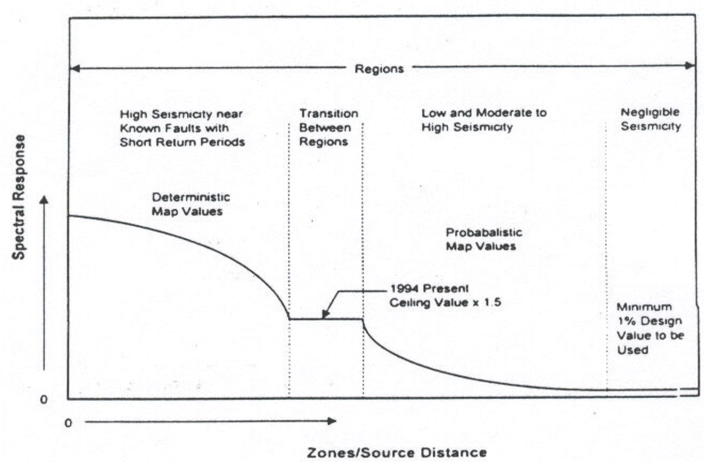

FIGURE A2 Development of the maximum considered earthquake ground motion map for spectral acceleration of T = 1.0, Site Class B.

STEP 1 -- DEFINE POTENTIAL SEISMIC SOURCES

A. ***Compile Earth Science Information*** **--** Compile historic seismicity and fault characteristics including earthquake magnitudes and recurrence intervals.

B. ***Prepare Seismic Source Map*** **--** Specify historic seismicity and faults used as sources.

STEP 2 -- PREPARE PROBABILISTIC AND DETERMINISTIC SPECTRAL RESPONSE MAPS

A. Develop Regional Attenuation Relations

(1) Eastern U.S. (Toro, et al., 1993, and Frankel, 1996)

(2) Western U.S. (Boore et al., 1993 &1994, Campbell and Bozorgnia, 1994, and Sadigh, 1993 for PGA. Boore et al., 1993 &1994, and Sadigh, 1993 for spectral values)

(3) Deep Events (>35km) (Geomatrix et al., 1993)

(4) Cascadia Subduction Zone (Geomatrix et al., 1993, and Sadigh, 1993)

B. ***Prepare Probabilistic Spectral Response Maps (USGS Probabilistic Maps)*** **--** Maps showing S_S and S_1 where S_S and S_1 are the short and 1 second period ground motion response spectral values for a 2 percent chance of exceedence in 50 years inferred for sites with average shear wave velocity of 760 m/s from the information developed in Steps 1A and 1B and the ground motion attenuation relationships in Step 2A.

C. ***Prepare Deterministic Spectral Response Maps (USGS Deterministic Maps)*** **--** Maps showing S_S and S_1 for faults and maximum earthquakes developed in Steps 1A and 1B and the median ground motion attenuation relations in Step 2A increased by 50% to represent the uncertainty.

STEP 3 -- PREPARE EARTHQUAKE GROUND MOTION SPECTRAL RESPONSE MAPS FOR PROVISIONS (MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION MAP)

Region 1 -- Regions of Negligible Seismicity with Very Low Probability of Collapse of the *Structure* (No Spectral Values)

Region definition: Regions for which S_S < 0.25g and S_1 < 0.10g from Step 2B.

Design values: No spectral ground motion values required. Use a minimum lateral force level of 1 percent of the dead load for Seismic Design Category A.

Region 2 -- Regions of Low and Moderate to High Seismicity (Probabilistic Map Values)

Region definition: Regions for which 0.25g < S_S < 1.5g and 0.25g < S_1 < 0.60g from Step 2B.

Maximum considered earthquake map values: Use S_S and S_I map values from Step 2B.

Transition Between Regions 2 and 3 - Use MCE values of S_S = 1.5g and S_1 = 0.60g

Region 3 -- Regions of High Seismicity Near Known Faults (Deterministic Values)

Region definition: Regions for which 1.5g < S_S and 0.60g < S_1 from Step 2C.

Maximum considered earthquake map values: Use S_S and S_I map values from Step 2C.

FIGURE A3 Methodology for development of the maximum considered earthquake ground motion maps (Site Class B).

Use of the Maximum Considered Earthquake Ground Motion Maps in the Design Procedure: The 1994 *Provisions* defined the seismic base shear as a function of the outdated effective peak velocity-related acceleration A_v, and effective peak acceleration, A_a. Beginning with the 1997 *Provisions*, the base shear of the structure is defined as a function of the maximum considered earthquake ground motion maps where S_S = maximum considered earthquake spectral acceleration in the short-period range for Site Class B; S_1 = maximum considered earthquake spectral acceleration at the 1.0 second period for Site Class B; $S_{MS} = F_a S_S$, maximum considered earthquake spectral acceleration in the short-period range adjusted for Site Class effects where F_a is the site coefficient defined in *Provisions* Sec. 4.1.2; $S_{M1} = F_v S_1$, maximum considered earthquake spectral acceleration at 1.0 second period adjusted for Site Class effects where F_v is the site coefficient defined in *Provisions* Sec. 4.1.2; S_{DS} = (2/3) S_{MS}, spectral acceleration in the short-period range for the design ground motions; and S_{D1} = (2/3) S_{M1}, spectral acceleration at 1.0 second period for the design ground motions.

As noted above, the design ground motions S_{DS} and S_{D1} are defined as 2/3 times the *maximum considered earthquake* ground motions. The 2/3 factor is based on the estimated seismic margins in the design process of the *Provisions* as previously discussed (i.e., the design level of ground motion is 1/1.5 or 2/3 times the maximum considered earthquake ground motion).

Based on the above defined ground motions, the base shear is:

$$V = C_s W$$

where $C_S = \frac{S_{DS}}{R/I}$ and S_{DS} = the design spectral response acceleration in the short period range as determined from Sec. 4.1.2, R = the response modification factor from Table 5.2.2, and I = the occupancy importance factor determined in accordance with Sec. 1.4.

The value of C_s need not exceed $C_S = \frac{S_{Dl}}{T\left(R/I\right)}$ but shall not be taken less than $C_S = 0.1 S_{Dl}$ or, for buildings and structures in Seismic Design Categories E and F, $C_S = \frac{0.5 S_1}{R/I}$

where I and R are as defined above and S_{D1} = the design spectral response acceleration at a period of 1.0 second as determined from Sec. 4.1.2, T = the fundamental period of the structure (sec) determined in Sec. 5.4.2, and S_1 = the mapped maximum considered earthquake spectral response acceleration determined in accordance with Sec. 4.1.

Where a design response spectrum is required by these *Provisions* and site-specific procedures are not used, the design response spectrum curve shall be developed as indicated in Figure A4 and as follows:

1. For periods less than or equal to T_0, the design spectral response acceleration, S_a, shall be taken as given by Eq. 4.1.2.6-1:

$$S_a = 0.6 \frac{S_{DS}}{T_0} T + 0.4 S_{DS} \qquad (4.1.2.6\text{-}1)$$

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response

3. For periods greater than T_S, the design spectral response acceleration, S_a, shall be taken as given by Eq. 4.1.2.6-3.

$$S_a = \frac{S_{Dl}}{T} \qquad (4.1.2.6\text{-}3)$$

where:

S_{DS} = the design spectral response acceleration at short periods;

S_{D1} = the design spectral response acceleration at 1 second period;

T = the fundamental period of the *structure* (sec);

T_0 = $0.2S_{D1}/S_{DS}$; and

T_S = S_{D1}/S_{DS}.

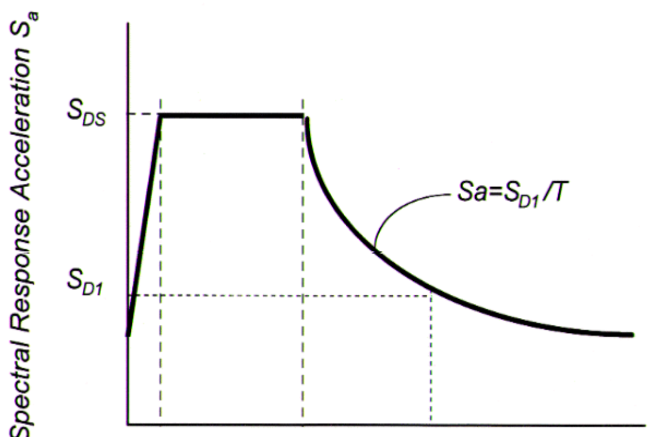

FIGURE A4 Design response spectrum.

Site-specific procedures for determining ground motions and response spectra are discussed in Sec. 4.1.3 of the *Provisions*.

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Commentary Appendix B

DEVELOPMENT OF THE USGS SEISMIC MAPS

INTRODUCTION

The 1997 *Provisions* used new design procedures based on the use of spectral response acceleration rather than the traditional peak ground acceleration and/or peak ground velocity, and these procedures are used again in the 2000 *Provisions*. The use of spectral ordinates and their relationship to building codes has been described by Leyendecker et al. (1995). The spectral response accelerations used in the new design approach are obtained from combining probabilistic maps (Frankel et al, 1996) prepared by the U.S. Geological Survey (USGS) with deterministic maps using procedures developed by the Building Seismic Safety Council's Seismic Design Procedures Group (SDPG). The SDPG recommendations are based on using the 1996 USGS probabilistic hazard maps with additional modifications based on review by the SDPG and the application of engineering judgment. This appendix summarizes the development of the USGS maps and describes how the 1997 and 2000 *Provisions* design maps were prepared from them using SDPG recommendations. The SDPG effort has sometimes been referred to as Project '97.

DEVELOPMENT OF PROBABILISTIC MAPS FOR THE UNITED STATES

New seismic hazard maps for the conterminous United States were completed by the USGS in June 1996 and placed on the World Wide Web (http://geohazards.cr.usgs.gov/eq/). The color maps can be viewed on the Web and/or downloaded to the user's computer for printing. Paper copies of the maps are also available (Frankel et al, 1997a, 1997b).

New seismic hazard maps for Alaska were completed by the USGS in January 1998 and placed on the USGS Web site (http://geohazards.cr.usgs.gov/eq/). Both documentation and printing of the maps are in progress (U. S. Geological Survey, 1998a, 1998b).

New probabilistic maps are in preparation for Hawaii using the methodology similar to that used for the rest of the United States, and described below. These maps were to have been completed in early 1998. Probabilistic maps for Puerto Rico, Culebra, Vieques, St. Thomas, St. John, St. Croix, Guam, and Tutuila needed for the 1997 *Provisions* are not expected during the current cycle of USGS map revisions (development of design maps for these areas is described below).

This appendix provides a brief description of the USGS seismic hazard maps, the geologic/seismologic inputs to these maps, and the ground-motion relations used for the maps. It is based on the USGS map documentation for the central and eastern United States (CEUS) and the western United States (WUS) prepared by Frankel et al. (1996). The complete reference document, also available on the USGS Web site, should be reviewed for detailed technical information.

The hazard maps depict probabilistic ground acceleration and spectral response acceleration with 10 percent, 5 percent, and 2 percent probabilities of exceedance (PE) in 50 years. These maps correspond to return times of approximately 500, 1000, and 2500 years, respectively.[1] All spectral response values shown in the maps correspond to 5 percent of critical damping. The maps are based on the assumption that earthquake occurrence is Poissonian, so that the probability of occurrence is time-

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[1] Previous USGS maps (e.g. Algermissen et al., 1990 and Leyendecker et al., 1995) and earlier editions of the *Provisions* expressed probability as a 10 percent probability of exceedance in a specified exposure time. Beginning with the 1996 maps, probability is being expressed as a specified probability of exceedance in a 50 year time period. Thus, 5 percent in 50 years and 2 percent in 50 years used now correspond closely to 10 percent in 100 years and 10 percent in 250 years, respectively, that was used previously. This same information may be conveyed as annual frequency. In this approach 10 percent probability of exceedance (PE) in 50 years corresponds to an annual frequency of exceedance of 0.0021; 5 percent PE in 100 years corresponds to 0.00103; and 2 percent PE in 50 years corresponds to 0.000404.

independent. The methodologies used for the maps were presented, discussed, and substantially modified during 6 regional workshops for the conterminous United States convened by the USGS from June 1994-June 1995. A seventh workshop for Alaska was held in September 1996.

The methodology for the maps (Frankel et al., 1996) includes three primary features:

1. The use of smoothed historical seismicity is one component of the hazard calculation. This is used in lieu of source zones used in previous USGS maps. The analytical procedure is described in Frankel (1995).
2. Another important feature is the use of alternative models of seismic hazard in a logic tree formalism. For the CEUS, different models based on different reference magnitudes are combined to form the hazard maps. In addition, large background zones based on broad geologic criteria are used as alternative source models for the CEUS and the WUS. These background zones are meant to quantify hazard in areas with little historic seismicity, but with the potential to produce major earthquakes. The background zones were developed from extensive discussions at the regional workshops.
3. For the WUS, a big advance in the new maps is the use of geologic slip rates to determine fault recurrence times. Slip rates from about 500 faults or fault segments were used in preparing the probabilistic maps.

The hazard maps do not consider the uncertainty in seismicity or fault parameters. Preferred values of maximum magnitudes and slip rates were used instead. The next stage of this effort is the quantification of uncertainties in hazard curves for selected sites. These data will be included on the Internet as they become available.

The USGS hazard maps are not meant to be used for Mexico, areas north of 49 degrees north latitude, and offshore the Atlantic and Gulf of Mexico coasts of the United States.

CEUS and WUS attenuation boundary. Attenuation of ground motion differs between the CEUS and the WUS. The boundary between regions was located along the eastern edge of the Basin and Range province (Figure B1). The previous USGS maps (e.g., Algermissen et al., 1990) used an attenuation boundary further to the east along the Rocky Mountain front.

Separate hazard calculations were done for the two regions using different attenuation relations. Earthquakes west of the boundary used the WUS attenuation relations and earthquakes east of the boundary used CEUS attenuation relations. WUS attenuation relations were used for WUS earthquakes, even for sites located east of the attenuation boundary. Similarly CEUS attenuations were used for CEUS earthquakes, even for sites located west of the attenuation boundary. It would have been computationally difficult to consider how much of the path was contained in the attenuation province. Also, since the attenuation relation is dependent on the stress drop, basing the relation that was used on the location of the earthquake rather than the receiver is reasonable.

Hazard curves. The probabilistic maps were constructed from mean hazard curves, that is the mean probabilities of exceedance as a function of ground motion or spectral response. Hazard curves were obtained for each site on a calculation grid.

A grid (or site) spacing of 0.1 degrees in latitude and longitude was used for the WUS and 0.2 degrees for the CEUS. This resulted in hazard calculations at about 65,000 sites for the WUS runs and 35,000 sites for the CEUS runs. The CEUS hazard curves were interpolated to yield a set of hazard curves on a 0.1 degree grid. A grid of hazard curves with 0.1 degree spacing was thereby obtained for the entire conterminous United States. A special grid spacing of 0.05 degrees was also done for California, Nevada, and western Utah because of the density of faults warranted increased density of data. These data were used for maps of this region.

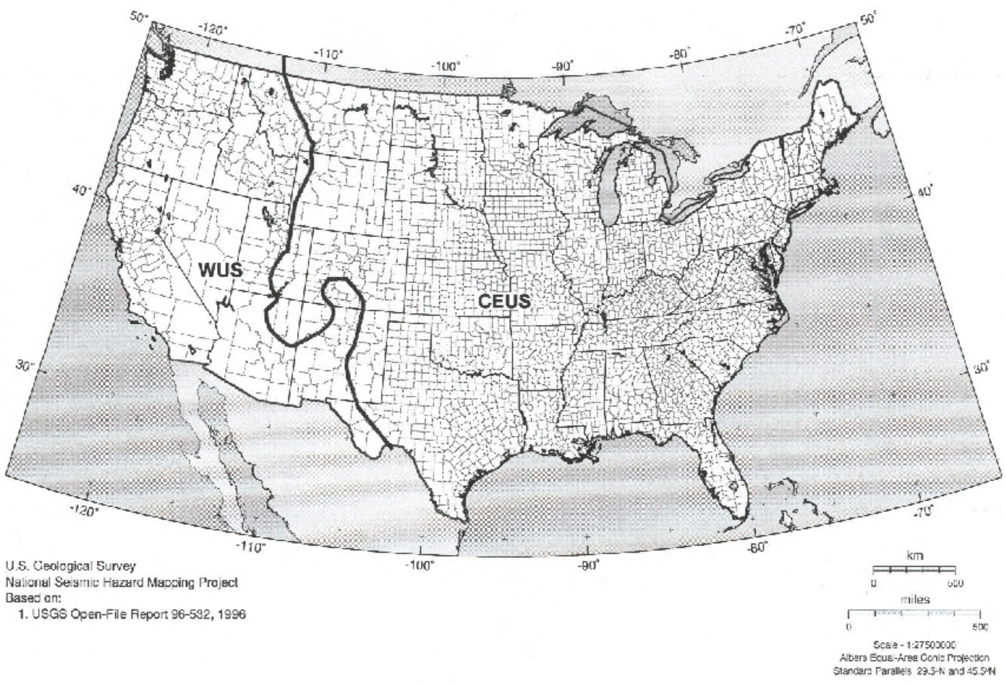

Figure B1 Attenuation boundary for eastern and western attenuation function.

Figure B2 is a sample of mean hazard curves used in making the 1996 maps. The curves include cities from various regions in the United States. It should be noted that in some areas the curves are very sensitive to the latitude and longitude selected. A probabilistic map is a contour plot of the ground motion or spectral values obtained by taking a “slice” through all 150,000 hazard curves at a particular probability value. The gridded data obtained from the hazard curves that was used to make each probabilistic map is located at the USGS Web site. Figure B2 also shows the general difference in slope of the hazard curves of the CEUS versus the WUS. This difference has been noted in other studies.

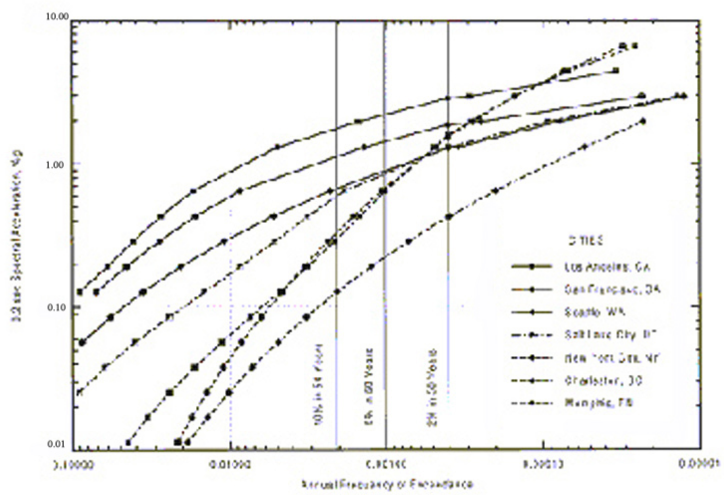

Figure B2 Hazard curves for selected cities.

CENTRAL AND EASTERN UNITED STATES

The basic procedure for constructing the CEUS portion of the hazard maps is diagrammed in Figure B3. Four models of hazard are shown on the left side of the figure. Model 1 is based on m_b 3.0 and larger earthquakes since 1924. Model 2 is derived from m_b 4.0 and larger earthquakes since 1860. Model 3 is produced from m_b 5.0 and larger events since 1700. In constructing the hazard maps, model 1 was assigned a weight twice that of models 2 and 3.

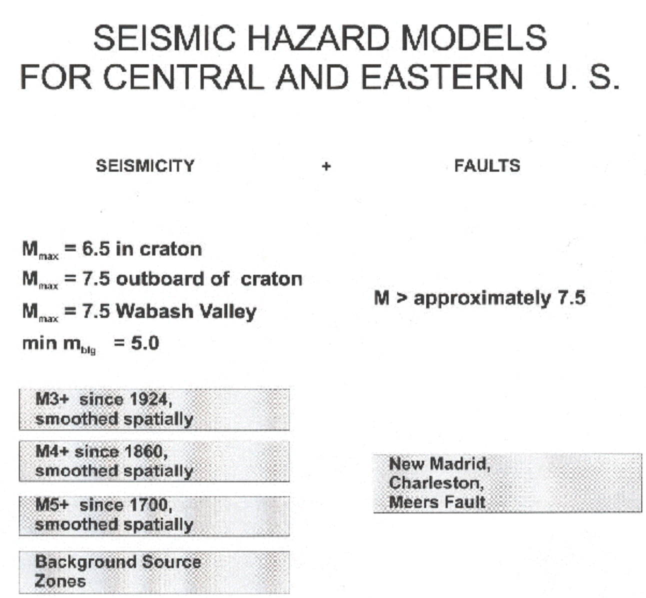

Figure B3 Seismic hazard models for the central and eastern United States. Smoothed seismicity models are shown on the left and fault models are shown on the right.

The procedure described by Frankel (1995) is used to construct the hazard maps directly from the historic seismicity (models 1-3). The number of events greater than the minimum magnitude are counted on a grid with spacing of 0.1 degrees in latitude and longitude. The logarithm of this number represents the maximum likelihood a-value for each grid cell. Note that the maximum likelihood method counts a m_b 5 event the same as a m_b 3 event in the determination of a-value. Then the gridded a-values are smoothed using a Gaussian function. A Gaussian with a correlation distance of 50 km was used for model 1 and 75 km for models 2 and 3. The 50 km distance was chosen because it is similar in width to many of the trends in historic seismicity in the CEUS. In addition, it is comparable to the error in location of m_b 3 events in the period of 1924-1975, before the advent of local seismic networks. A larger correlation distance was used for models 2 and 3 since they include earthquakes further back in time with poorer estimates of locations.

Model 4 consists of large background source zones. This alternative is meant to quantify hazard in areas with little historical seismicity but with the potential to generate damaging earthquakes. These background zones are detailed in a later section of this text. The sum of the weights of models 1-4 is one. For a weighting scheme that is uniform in space, this ensures that the total seismicity rate in the combined model equals the historic seismicity rate. A spatially-varying weighting scheme which slightly exceeds the historic seismicity rate was used in the final map for reasons which are described later.

A regional b-value of 0.95 was used for models 1-4 in all of the CEUS except Charlevoix, Quebec. This b-value was determined from a catalog for events east of 105 degrees W. For the Charlevoix region a b-value of 0.76 was used based on the work of John Adams, Stephen Halchuck and Dieter Weichert of the Geologic Survey of Canada (see Adams et al., 1996).

Figure B4 shows a map of the CEUS M_{max} values used for models 1-4 (bold M refers to moment magnitude). These M_{max} zones correspond to the background zones used in model 4. Most of the CEUS is divided into a cratonic region and a region of extended crust. An M_{max} of 6.5 was used for the cratonic area. A M_{max} of 7.5 was used for the Wabash Valley zone in keeping with magnitudes derived from paleoliquefaction evidence (Obermeier et al., 1992). An M_{max} of 7.5 was used in the zone of extended crust outboard of the craton. An M_{max} of 6.5 was used for the Rocky Mountain zone and the Colorado Plateau, consistent with the magnitude of the largest historic events in these regions. An M_{max} of 7.2 was used for the gridded seismicity within the Charleston areal source zone. A minimum m_b of 5.0 was used in all the hazard calculations for the CEUS.

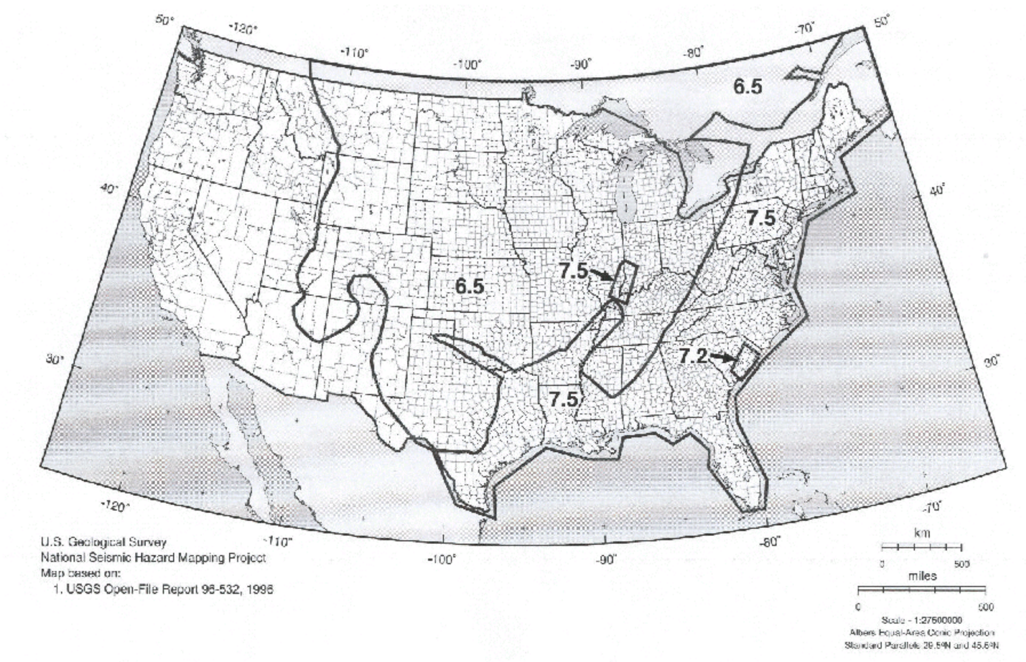

Figure B4 Central and eastern U.S. maximum magnitude zones.

Model 5 (Figure B3, right) consists of the contribution from large earthquakes (M > 7.0) in four specific areas of the CEUS: New Madrid, Charleston, South Carolina, the Meers fault in southwest Oklahoma, and the Cheraw Fault in eastern Colorado. This model has a weight of 1. The treatment of these special areas is described below. There are three other areas in the CEUS that are called special zones: eastern Tennessee, Wabash Valley, and Charlevoix. These are described below.

Special zones A number of special case need to be described.

New Madrid: To calculate the hazard from large events in the New Madrid area, three parallel faults in an S-shaped pattern encompassing the area of highest historic seismicity were considered. These were not meant to be actual faults; they are simply a way of expressing the uncertainty in the source locations of large earthquakes such as the 1811-12 sequence. A characteristic rupture model with a characteristic moment magnitude M of 8.0, similar to the estimated magnitudes of the largest events in 1811-12 (Johnston, 1996a, 1996b) was assumed. A recurrence time of 1000 years for such an event was used as an average value, considering the uncertainty in the magnitudes of pre-historic events.

An areal source zone was used for New Madrid for models 1-3, rather than spatially-smoothed historic seismicity. This zone accounts for the hazard from New Madrid events with moment magnitudes less than 7.5.

Charleston, South Carolina: An areal source zone was used to quantify the hazard from large earthquakes. The extent of the areal source zone was constrained by the areal distribution of paleoliquefaction locations, although the source zone does not encompass all the paleoliquefaction sites. A characteristic rupture model of moment magnitude 7.3 earthquakes, based on the estimated magnitude of the 1886 event (Johnston, 1996b) was assumed. For the M7.3 events a recurrence time of 650 years was used, based on dates of paleoliquefaction events (Amick and Gelinas, 1991; Obermeier et al., 1990; Johnston and Schweig, written comm., 1996).

Meers Fault: The Meers fault in southwestern Oklahoma was explicitly included. The segment of the fault which has produced a Holocene scarp as described in Crone and Luza (1990) was used. A characteristic moment magnitude of 7.0 and a recurrence time of 4000 years was used based on their work.

Cheraw Fault: This eastern Colorado fault with Holocene faulting based on a study by Crone et al. (1996) was included. The recurrence rate of this fault was obtained from a slip rate of 0.5 mm/yr. A maximum magnitude of 7.1 was found from the fault length using the relations of Wells and Coppersmith (1994).

Eastern Tennessee Seismic Zone: The eastern Tennessee seismic zone is a linear trend of seismicity that is most obvious for smaller events with magnitudes around 2 (see Powell et al., 1994). The magnitude 3 and larger earthquakes tend to cluster in one part of this linear trend, so that hazard maps are based just on smoothed m_b3.

Wabash Valley: Recent work has identified several paleoearthquakes in the areas of southern Indiana and Illinois based on widespread paleoliquefaction features (Obermeier et al., 1992). An areal zone was used with a higher M_{max} of 7.5 to account for such large events. The sum of the gridded a-values in this zone calculated from model 1 produce a recurrence time of 2600 years for events with m_b 6.5. The recurrence rate of M6.5 and greater events is estimated to be about 4,000 years from the paleoliquefaction dates (P. Munson and S. Obermeier, pers. comm., 1995), so it is not necessary to add additional large events to augment models 1-3. The Wabash Valley M_{max} zone in the maps is based on the Wabash Valley fault zone.

Charlevoix, Quebec: As mentioned above, a 40 km by 70 km region surrounding this seismicity cluster was assigned a b-value of 0.76, based on the work of Adams, Halchuck and Weichert. This b-value was used in models 1-3.

Background source zones (Model 4). The background source zones (see Figure B5) are intended to quantify seismic hazard in areas that have not had significant historic seismicity, but could very well produce sizeable earthquakes in the future. They consist of a cratonic zone, an extended margin zone, a Rocky Mountain zone, and a Colorado Plateau zone. The Rocky Mountain zone was not discussed at any workshop, but is clearly defined by the Rocky Mountain front on the east and the areas of extensional tectonics to the west, north and south. As stated above, the dividing line between the cratonic and extended margin zone was drawn by Rus Wheeler based on the westward and northern edge of rifting during the opening of the Iapetan ocean. One justification for having craton and extended crust zones is the work done by Johnston (1994). They compiled a global survey of earthquakes in cratonic and extended crust and found a higher seismicity rate (normalized by area) for the extended areas.

For each background zone, a-values were determined by counting the number of m_b3 and larger events within the zone since 1924 and adjusting the rate to equal that since 1976. A b-value of 0.95 was used for all the background zones, based on the b-value found for the entire CEUS.

Adaptive weighting for CEUS. The inclusion of background zones lowers the probabilistic ground motions in areas of relatively high historic seismicity while raising the hazard to only low levels in areas with no historic seismicity. The June 1996 versions of the maps include the background zones using a weighting scheme that can vary locally depending on the level of historic seismicity in that cell of the a-value grid. Spatially-varying weighting was suggested by Allin Cornell in the external review of the interim maps. The “adaptive weighting” procedure avoids lowering the hazard in higher seismicity areas to raise the hazard in low seismicity areas. This was implemented by looping through the a-value grid and checking to see if the a-value for each cell from the historic seismicity was greater than the a-value from the background zone. For the CEUS the a-value from the historic seismicity was derived by weighting the rates from models 1, 2, and 3 by 0.5, 0.25, 0.25 respectively. If this weighted sum was greater than the rate from the appropriate background zone, then the rate for that cell was determined by weighting the rates from models 1-3 by 0.5, .25, .25 (i.e., historic seismicity only, no background zone). If the weighted sum from the historic seismicity was less than the rate of the background zone, then a weighting of 0.4, 0.2, 0.2, 0.2 for models 1-4, respectively (including the background zone as model 4). This procedure does not make the rate for any cell lower than it would be from the historic seismicity (models 1-3). It also incorporates the background zones in areas of low historic seismicity. The total seismicity rate in the resulting a-value grid is only 10 percent larger than the observed rate of m_b3’s since 1976. This is not a major difference. Of course, this procedure produces substantially higher ground motions (in terms of percentage increase) in the seismically quiet areas as compared to no background zone. These values are still quite low in an absolute sense.

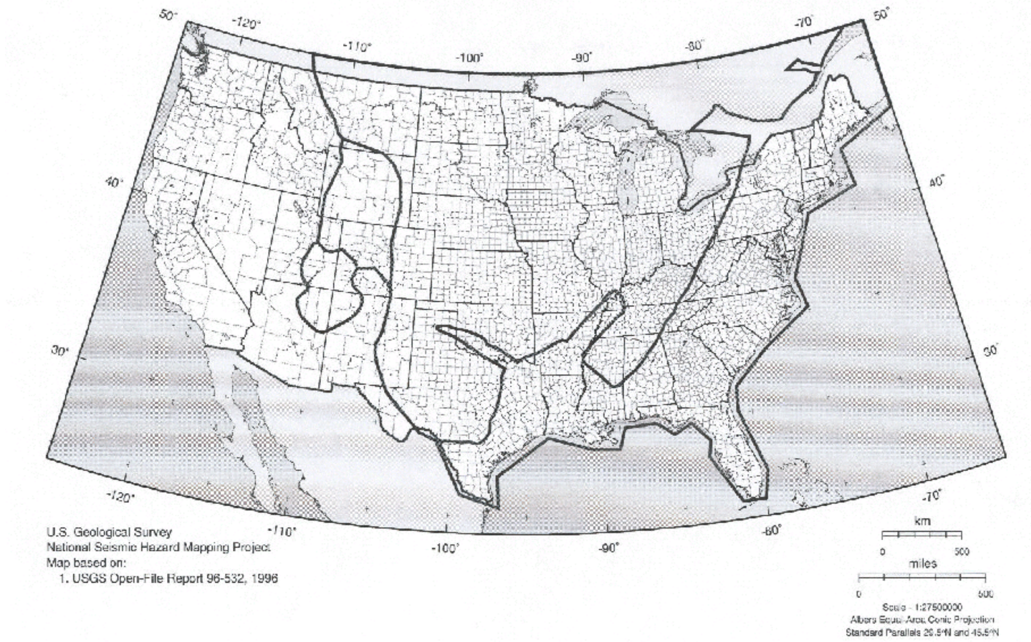

Figure B5 Central and eastern U.S. background zones.

CEUS catalogs and b-value calculation. The primary catalog used for the CEUS for longitudes east of 105 degrees is Seeber and Armbruster (1991), which is a refinement of the EPRI (1986) catalog. This was supplemented with the PDE catalog from 1985-1995. In addition, PDE, DNAG, Stover and Coffman (1993), Stover, Reagor, and Algermissen (1984) catalogs were searched to find events not included in Seeber and Armbruster (1991). Mueller et al. (1996) describes the treatment of catalogs, adjustment of rates to correct for incompleteness, the removal of aftershocks, and the assignment of magnitudes.

Attenuation relations for CEUS. The reference site condition used for the maps is specified to be the boundary between *Provisions* Site Classes B and C (Martin and Dobry, 1994), meaning it has an average shear-wave velocity of 760 m/sec in the top 30m. This corresponds to a typical "firm-rock" site for the western United States (see WUS attenuation section below), although many rock sites in the CEUS probably have much higher velocities. The motivation for using this reference site is that it corresponds to the average of sites classified as "rock" sites in WUS attenuation relations. In addition, it was considered less problematic to use this site condition for the CEUS than to use a soil condition. Most previously-published attenuation relations for the CEUS are based on a hard-rock site condition. It is less of a problem to convert these to a firm-rock condition than to convert them to a soil condition, since there would be less concern over possible non-linearity for the firm-rock site compared to the soil site.

Two equally-weighted, attenuation relations were used for the CEUS. Both sets of relations were derived by stochastic simulations and random vibration theory. First the Toro et al. (1993) attenuation for hard-rock was used. The attenuation relations were multiplied by frequency-dependent factors developed by USGS to convert them from hard-rock to firm-rock sites. The factors used 1.52 for PGA, 1.76 for 0.2 sec spectral response, 1.72 for 0.3 sec spectral response and 1.34 for 1.0 sec spectral response. These factors were applied independently of magnitude and distance.

The second set of relations was derived by USGS (Frankel et al., 1996) for firm-rock sites. These relations were based on a Brune source model with a stress drop of 150 bars. The simulations contained frequency-dependent amplification factors derived from a hypothesized shear-wave velocity profile of a CEUS firm-rock site. A series of tables of ground motions and response spectral values as a function of moment magnitude and distance was produced instead of an equation.

For CEUS hazard calculations for models 1-4, a source depth of 5.0 km was assumed when using the USGS ground motion tables. Since a minimum hypocentral distance of 10 km is used in the USGS tables, the probabilistic ground motions are insensitive to the choice of source depth. In the hazard program, when hypocentral distances are less than 10 km the distance is set to 10 km when using the tables. For the Toro et al. (1993) relations, the fictitious depths that they specify for each period are used, so that the choice of source depth used in the USGS tables was not applied.

For both sets of ground motion relations, values of 0.75, 0.75, 0.75, and 0.80 were used for the natural logarithms of the standard deviation of PGA, 0.2 sec, 0.3 sec, and 1.0 sec spectral responses, respectively. These values are similar to the aleatory standard deviations reported to the Senior Seismic Hazard Analysis Committee (1996).

A cap in the median ground motions was placed on the ground motions within the hazard code. USGS was concerned that the median ground motions of both the Toro et al. and the new USGS tables became very large (> 2.5g PGA) for distances of about 10 km for the M 8.0 events for New Madrid. Accordingly the median PGA's was capped at 1.5g. The median 0.3 and 0.2 sec values were capped at 3.75g which was derived by multiplying the PGA cap by 2.5 (the WUS conversion factor). This only affected the PGA values for the 2 percent PE in 50 year maps for the area directly above the three fictitious faults for the New Madrid region. It does not change any of the values at Memphis. The capping did not significantly alter the 0.3 and 0.2 sec values in this area. The PGA and spectral response values did not change in the Charleston region from this capping. Note that the capping was for the median values only. As the variability (sigma) of the ground motions was maintained in the hazard code, values larger than the median were allowed. USGS felt that the capping recognizes that values derived from point source simulations are not as reliable for M8.0 earthquakes at close-in distances (< 20 km).

Additional notes for CEUS. One of the major outcomes of the new maps for the CEUS is that the ground motions are about a factor of 2 to 3 times lower, on average, than the PGA values in Algermissen et al. (1990) and the spectral values in Algermissen et al. (1991) and Leyendecker et al. (1995). The primary cause of this difference is the magnitudes assigned to pre-instrumental earthquakes in the catalog. Magnitudes of historic events used by Algermissen et al. were based on I_{max} (maximum observed intensity), using magnitude-I_{max} relations derived from WUS earthquakes. This overestimates the magnitudes of these events and, in turn, overestimates the rates of M4.9 and larger events. The magnitudes of historic events used in the new maps were primarily derived by Seeber and Armbruster (1991) from either felt area or I_{max} using relations derived from CEUS earthquakes (Sibol et al., 1987). Thus, rates of M4.9 and larger events are much lower in the new catalog, compared to those used for the previous USGS maps.

It is useful to compare the new maps to the source zones used in the EPRI (1986) study. For the areas to the north and west of New Madrid, most of the six EPRI teams had three source zones in common: 1) the Nemaha Ridge in Kansas and Nebraska, 2) the Colorado-Great Lakes lineament extending from Colorado to the western end of Lake Superior, and 3) a small fault zone in northern Illinois, west of Chicago. Each of these source zones are apparent as higher hazard areas in the our maps. The Nemaha Ridge is outlined in the maps because of magnitude 4 and 5 events occurring in the vicinity. Portions of the Colorado-Great Lakes lineament show higher hazard in the map, particularly the portion in South Dakota and western Minnesota. The portion of the lineament in eastern Minnesota has been historically inactive, so is not apparent on the maps. The area in western Minnesota shows some hazard because of the occurrence of a few magnitude 4 events since 1860. A recent paper by Chandler (1995), argues that the locations and focal mechanisms of these earthquakes are not compatible with them being on the lineament, which is expressed as the Morris Fault in this region. The area in northern Illinois has relatively high hazard in the maps because of M4-5 events that have occurred there.

Frankel (1995) also found good agreement in the mean PE's and hazard curves derived from models 1-3 and 4 and those produced by the EPRI (1986) study, when the same PGA attenuation relations were used.

WESTERN UNITED STATES

The maps for the WUS include a cooperative effort with the California Division of Mines and Geology. This was made possible, in part, because CDMG was doing a probabilistic map at the same time the USGS maps were prepared. There was considerable cooperation in this effort. For example, the fault data base used in the USGS maps was obtained from CDMG. Similarly USGS software was made available to CDMG. The result is that maps produced by both agencies are the same.

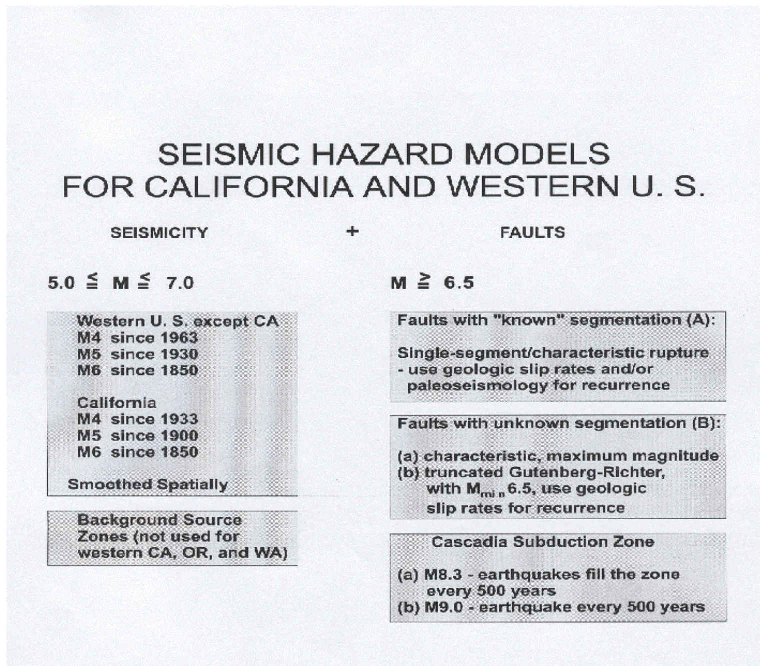

Figure B6 Seismic hazard models for California and the western United States. Smoothed seismicity models are shown on the left and fault models are shown on the right.

The procedure for mapping hazard in the WUS is shown in Figure B6. On the left side, hazards are considered from earthquakes with magnitudes less than or equal to moment magnitude 7.0. For most of the WUS, two alternative models are used: 1) smoothed historical seismicity (weight of 0.67) and 2) large background zones (weight 0.33) based on broad geologic criteria and workshop input. Model 1 used a 0.1 degree source grid to count number of events. The determination of a-value was changed somewhat from the CEUS, to incorporate different completeness times for different magnitude ranges. The a-value for each grid cell was calculated from the maximum likelihood method of Weichert (1980), based on events with magnitudes of 4.0 and larger. The ranges used were M4.0 to 5.0 since 1963, M5.0 to 6.0 since 1930, and M6.0 and larger since 1850. For the first two categories, completeness time was derived from plots of cumulative number of events versus time. M3 events were not used in the WUS hazard calculations since they are only complete since about 1976 for most areas and may not even be complete after 1976 for some areas. For California M4.0 to M5.0 since 1933, M5.0 to 6.0 since 1900, and M6.0 and larger since 1850 were used. The catalog for California is complete to earlier dates compared to the catalogs for the rest of the WUS (see below).

Another difference with the CEUS is that multiple models with different minimum magnitudes for the a-value estimates (such as models 1-3 for the CEUS) were not used. The use of such multiple models in the CEUS was partially motivated by the observation that some m_b4 and m_b5 events in the CEUS occurred in areas with few m_b3 events since 1924 (e.g., Nemaha Ridge events and western Minnesota events). It was considered desirable to be able to give such m_b4 and m_b5 events extra weight in the hazard calculation over what they would have in one run with a minimum magnitude of 3. In contrast it appears that virtually all M5 and M6 events in the WUS have occurred in areas with numerous M4 events since 1965. There was also reluctance to use a WUS model with a-values based on a minimum magnitude of 6.0, since this would tend to double count events that have occurred on mapped faults included in Figure B6 right.

For model 1, the gridded a-values were smoothed with a Gaussian with a correlation distance of 50 km, as in model 1 for the CEUS. The hazard calculation from the gridded a-values differed from that in the CEUS, because we considered fault finiteness in the WUS calculations. For each source grid cell, a fictitious fault for magnitudes of 6.0 and larger was used. The fault was centered on the center of the grid cell. The strike of the fault was random and was varied for each magnitude increment. The length of the fault was determined from the relations of Wells and Coppersmith (1994). The fictitious faults were taken to be vertical.

A maximum moment magnitude of 7.0 was used for models 1 and 2, except for four shear zones in northeastern California and western Nevada described below. Of course, larger moment magnitudes are included in the specific faults. A minimum moment magnitude of 5.0 were used for models 1 and 2. For each WUS site, the hazard calculation was done for source-site distances of 200 km and less, except for the Cascadia subduction zone, where the maximum distance was 1000 km.

Separate hazard calculations for deep events (> 35 km) were done. These events were culled from the catalogs. Their a-values were calculated separately from the shallow events. Different attenuation relations were used.

Regional b-values were calculated based on the method of Weichert (1980), using events with magnitudes of 4 and larger and using varying completeness times for different magnitudes. Accordingly, a regional b-value of 0.8 was used in models 1 and 2 for the WUS runs based on shallow events. For the deep events (> 35 km), an average b-value of 0.65 was found. This low b-value was used in the hazard calculations for the deep events.

We used a b-value of 0.9 for most of California, except for the easternmost portion of California in our basin and range background zone (see below). This b-value was derived by CDMG.

Faults. The hazard from about 500 Quaternary faults or fault segments was used for the maps. Faults were considered where geologic slip rates have been determined or estimates of recurrence times have been made from trenching studies. A table of the fault parameters used in the hazard calculations has been compiled and is shown on the USGS Internet Web site. Figure B7 shows the faults used in the maps. The numerous individuals who worked on compilations of fault data are too numerous to cite here. They are cited, along with their contribution, in the map documentation (Frankel et al, 1996).

Recurrence models for faults. The hazard from specific faults is added to the hazard from the seismicity as shown in Figure B6. Faults are divided into types A and B, roughly following the nomenclature of WGCEP (1995). A fault is classified as A-type if there have been sufficient studies of it to produce models of fault segmentation. In California the A-type faults are: San Andreas, San Jacinto, Elsinore, Hayward, Rodgers Creek, and Imperial (M. Petersen, C. Cramer, and W. Bryant, written comm., 1996). The only fault outside of California classified as an A-type is the Wasatch Fault. Single-segment ruptures were assumed on the Wasatch Fault.

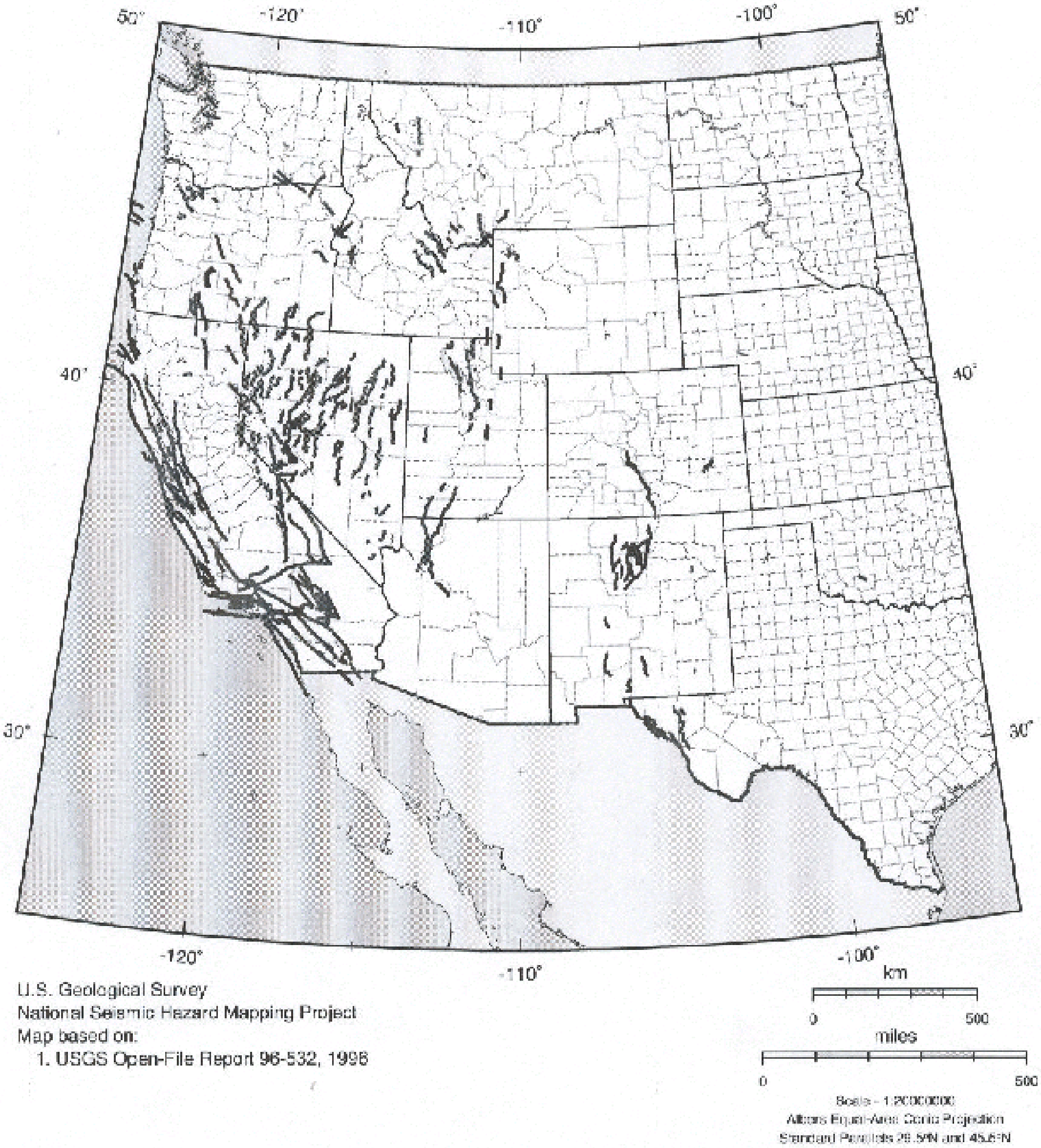

Figure B7 Western U.S. faults included in the maps.

For California, the rupture scenarios specified by Petersen, Cramer and Bryant of CDMG, with input from Lienkaemper of USGS for northern California were used. Single-segment, characteristic rupture for the San Jacinto and Elsinore faults were assumed. For the San Andreas fault, multiple-segment ruptures were included in the hazard calculation, including repeats of the 1906 and 1857 rupture zones, and a scenario with the southern San Andreas fault rupturing from San Bernardino through the Coachella segment. Both single-segment and double-segment ruptures of the Hayward Fault were included.

For California faults, characteristic magnitudes derived by CDMG from the fault area using the relations in Wells and Coppersmith (1994) were used. For the remainder of the WUS, the characteristic magnitude was determined from the fault length using the relations of Wells and Coppersmith (1994) appropriate for that fault type.

For the B-type faults, it was felt there were insufficient studies to warrant specific segmentation boundaries. For these faults, the scheme of Petersen et al. (1996) was followed, using both characteristic and Gutenberg-Richter (G-R; exponential) models of earthquake occurrence. These recurrence models were weighted equally. The G-R model basically accounts for the possibility that a fault is segmented and may rupture only part of its length. It was assumed that the G-R distribution applies from a minimum moment magnitude of 6.5 up to a moment magnitude corresponding to rupture of the entire fault length.

The procedure for calculating hazard using the G-R model involves looping through magnitude increments. For each magnitude a rupture length is calculated using Wells and Coppersmith (1994). Then a rupture zone of this length is floated along the fault trace. For each site, the appropriate distance to the floating ruptures is found and the frequency of exceedance (FE) is calculated. The FE's are then added for all the floating rupture zones.

As used by USGS, the characteristic earthquake model (Schwartz and Coppersmith, 1984) is actually the maximum magnitude model of Wesnousky (1986). Here it is assumed that the fault only generates earthquakes that rupture the entire fault. Smaller events along the fault would be incorporated by models 1 and 2 with the distributed seismicity or by the G-R model described above.

It should be noted that using the G-R model generally produces higher probabilistic ground motions than the characteristic earthquake model, because of the more frequent occurrence of earthquakes with magnitudes of about 6.5.

Fault widths (except for California) were determined by assuming a seismogenic depth of 15 km and then using the dip, so that the width equaled 15 km divided by the sine of the dip. For most normal faults a dip of 60 degrees is assumed. Dip directions were taken from the literature. For the Wasatch, Lost River, Beaverhead, Lemhi, and Hebgen Lake faults, the dip angles were taken from the literature (see fault parameter table on Web site). Strike-slip faults were assigned a dip of 90 degrees. For California faults, widths were often defined using the depth of seismicity (J. Lienkaemper, written comm., 1996; M. Petersen, C. Cramer, and W. Bryant, written comm., 1996). Fault length was calculated from the total length of the digitized fault trace.

Special cases. There are a number of special cases which need to be described.

Blind thrusts in the Los Angeles area: Following Petersen et al. (1996) and as discussed at the Pasadena workshop, 0.5 weight was assigned to blind thrusts in the L.A. region, because of the uncertainty in their slip rates and in whether they were indeed seismically active. These faults are the Elysian Park thrust and the Compton thrust. The Santa Barbara Channel thrust (Shaw and Suppe, 1994) also has partial weight, based on the weighting scheme developed by CDMG.

Offshore faults in Oregon: A weight of 0.05 was assigned to three offshore faults in Oregon identified by Goldfinger et al. (in press) and tabulated by Geomatrix (1995): the Wecoma, Daisy Bank and Alvin Canyon faults. It was felt the uncertainty in the seismic activity of these faults warranted a low weight, and the 0.05 probability of activity decided in Geomatrix (1995) was used. A 0.5 weight was assigned to the Cape Blanco blind thrust.

Lost River, Lemhi and Beaverhead faults in Idaho: It was assumed that the magnitude of the Borah Peak event (M7.0) represented a maximum magnitude for these faults. As with (3), the characteristic model floated a M7.0 along each fault. The G-R model considered magnitudes between 6.5 and 7.0. Note that using a larger maximum magnitude would lower the probabilistic ground motions, because it would increase the recurrence time.

Hurricane and Sevier-Torroweap Faults in Utah and Arizona: The long lengths of these faults (about 250 km) implied a maximum magnitude too large compared to historical events in the region. Therefore a maximum magnitude of M7.5 was chosen. The characteristic and G-R models were implemented as in case (3). Other faults (outside of California) where the M_{max} was determined to be greater than 7.5 based on the fault length were assigned a maximum magnitude of 7.5.

Wasatch Fault in Utah: Recurrence times derived from dates of paleoearthquakes by Black et al. (1995) and the compilation of McCalpin and Nishenko (1996) were used

Hebgen Lake Fault in Montana: A characteristic moment magnitude of 7.3 based on the 1959 event (Doser, 1985) was used.

Short faults: All short faults with characteristic magnitudes of less than 6.5 were treated with the characteristic recurrence model only (weight = 1). No G-R relation was used. If a fault had a characteristic magnitude less than 6.0, it was not used.

Seattle Fault: The characteristic recurrence time was fixed at 5000 years, which is the minimum recurrence time apparent from paleoseismology (R. Bucknam, pers. comm., 1996). Using the characteristic magnitude of 7.1 derived from the length and a 0.5 mm/yr slip rate yielded a characteristic recurrence time of about 3000 years.

Eglington fault near Las Vegas: The recurrence time for this fault was fixed at 14,000 years, similar to the recurrence noted in Wyman et al. (1993).

Shear Zones in Eastern California and Western Nevada: Areal shear zones were added along the western border of Nevada extending from the northern end of the Death Valley fault through the Tahoe-Reno area through northeast California ending at the latitude of Klamath Falls, Oregon. A shear rate of 4 mm/yr to zone 1, and 2 mm/yr to zones 2 and 3 was assigned. The shear rate in zone 1 is comparable to the shear rate observed on the Death Valley fault, but which is not observed in mapped faults north of the Death Valley fault (C. dePolo and J. Anderson, pers. comm., 1996). For the Foothills Fault system (zone 4) a shear rate of 0.05 mm/yr was used. a-values were determined for these zones in the manner described in Ward(1994). For zones 1 through 3, a magnitude range of 6.5 to 7.3 was used. For zone 4, a magnitude range of 6.0 to 7.0 was used. The maximum magnitude for the calculation of hazard from the smoothed historic seismicity was lowered in these zones so that it did not overlap with these magnitude ranges. Fictitious faults with a fixed strike were used in the hazard calculation for these zones. Again, use of these areal zones in California was agreed upon after consultation with CDMG personnel.

Cascadia subduction zone. Two alternative scenarios for great earthquakes on the Cascadia subduction zone were considered. For both scenarios it was assumed that the recurrence time of rupture at any point along the subduction zone was 500 years. This time is in or near most of the average intervals estimated from coastal and offshore evidence (see Atwater and Hemphill-Haley, 1996; Geomatrix, 1995; B. Atwater, written comm., 1996). Individual intervals, however, range from a few hundred years to about 1000 years (Atwater et al., 1995).

The first scenario is for moment magnitude 8.3 earthquakes to fill the subduction zone every 500 years. Based on a rupture length of 250 km (see Geomatrix, 1995) for an M8.3 event and the 1100 km length of the entire subduction zone, this requires a repeat time of about 110 years for an M8.3 event. However, no such event has been observed in the historic record of about 150 years. This M8.3 scenario is similar to what was used in the 1994 edition of the USGS maps (see Leyendecker et al., 1995) and it is comparable to the highest weighted scenario in Geomatrix (1995). A M8.3 rupture zone was floated along the strike of the subduction zone to calculate the hazard. A weight of 0.67 was assigned for this scenario in the maps.

The second scenario used is for a moment magnitude 9.0 earthquake to rupture the entire Cascadia subduction zone every 500 years on average. No compelling reason was seen to rule out such a scenario. This scenario would explain the lack of M8 earthquakes in the historic record. It is also consistent with a recent interpretation of Japanese tsunami records by Satake et al. (1996). By ruling out alternative source regions, Satake et al. (1996) reported that a tsunami in 1700 could have been produced by a M9.0 earthquake along the Cascadia subduction zone. A weight of 0.33 was assigned to the M9.0 scenario in the maps.

The subduction zone was specified as a dipping plane striking north-south from about Cape Mendocino to 50 degrees north. It was assumed that the plane reached 20 km depth at a longitude of

123.8 degrees west, just east of the coastline. This corresponds roughly to the 20 km depth contour drawn by Hyndman and Wang (1995) and is consistent with the depth and location of the Petrolia earthquake in northern California. A dip of 10 degrees was assigned to the plane and a width of 90 km. The seismogenic portion of the plane was assumed to extend to a depth of 20 km.

Background source zones. The background source zones for the WUS (model 2) were based on broad geologic criteria and were developed by discussion at the Salt Lake City (SLC) workshop (except for the Cascades source zone). These zones are shown in Figure B8. Note that there are no background source zones west of the Cascades and west of the Basin and Range province. For those areas, model 1 was used with a weight of 1.

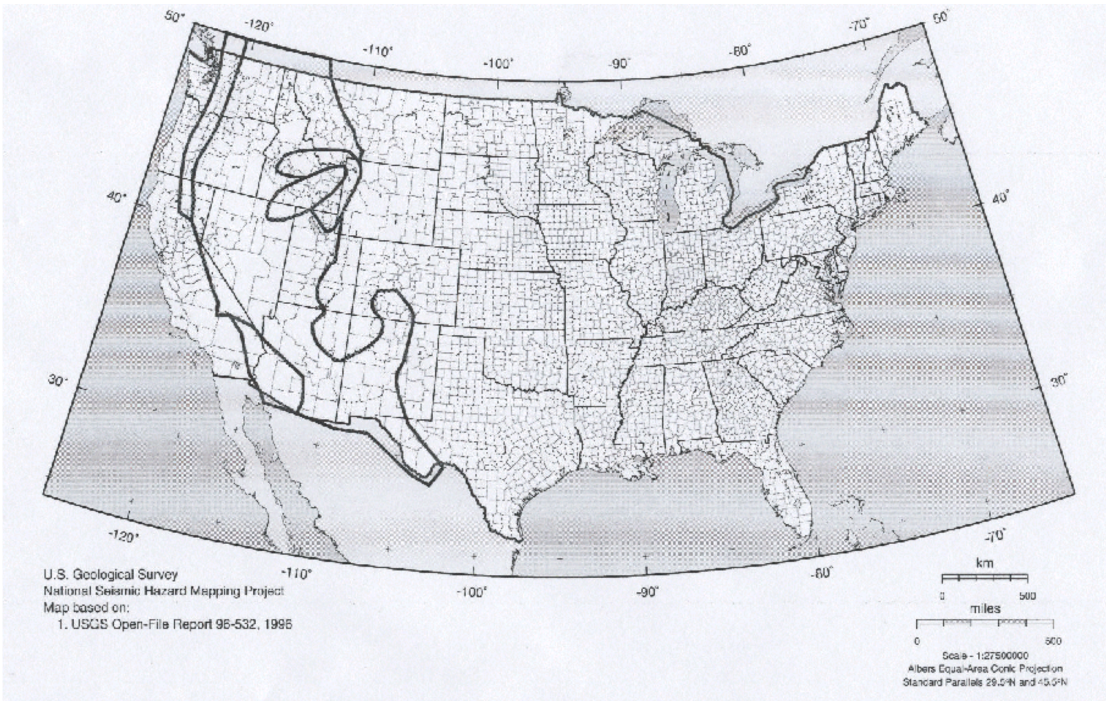

Figure B8 Western U.S. background zones.

At the SLC workshop there was substantial sentiment for a Yellowstone Parabola source zone (see, e.g., Anders et al., 1989) that would join up seismically-active areas in western Wyoming with the source areas of the Bora Peak and Hebgen Lake earthquakes. It was felt that the relatively seismically-quiescent areas consisting of the Snake River Plain and Colorado Plateau should be separate source zones because of the geologic characteristics. An area of southwest Arizona was suggested as a separate source zone by Bruce Scheol, based partly on differences in the age and length of geologic structures compared with the Basin and Range Province (see Edge et al., 1992). A Cascades source zone was added since it was felt that was a geologically-distinct area.

The remaining background source zone includes the Basin and Range Province, the Rio Grande Rift, areas of Arizona and New Mexico, portions of west Texas, and areas of eastern Washington and northern Idaho and Montana. The northern border of this zone follows the international border. As stated above, this seems to be a valid approach since the hazard maps are being based on the seismicity rate in the area of interest.

This large background zone is intended to address the possibility of having large earthquakes (M6 and larger) in areas with relatively low rates of seismicity in the brief historic record. It is important to have a large zone that contains areas of high seismicity in order to quantify the hazard in relatively quiescent areas such as eastern Oregon and Washington, central Arizona, parts of New Mexico, and west Texas. One can see the effect of this large background zone by noting the contours on the hazard maps in these areas. The prominence of the background zones in the maps is determined by the weighting of models 1 and 2.

Adaptive weighting for the WUS. The adaptive weighting procedure was used to include the background zones in the WUS without lowering the hazard values in the high seismicity areas. As with the CEUS, the a-value was checked for each source cell to see whether the rate from the historic seismicity exceeded that from the appropriate background zone. If it did, the a-value was used from the historic seismicity. If the historic seismicity a-value was below the background value, then a rate derived from using 0.67 times the historic rate plus 0.33 times the background rate was used. This does not lower the a-value in any cell lower than the value from the historic seismicity. The total seismicity rate in this portion of the WUS in the new a-value grid is 16 percent above the historic rate (derived from M4 and greater events since 1963).

WUS catalogs. For the WUS, except for California, the Stover and Coffman (1993), Stover, Reagor, and Algermissen (1984), PDE, and DNAG catalogs (with the addition of Alan Sanford's catalog for New Mexico) were used. For California, a catalog compiled by Mark Petersen of California Division of Mines and Geology (CDMG) was used. Mueller et al. (1996) describes the processing of the catalogs, the removal of aftershocks, and the assignment of magnitudes. Utah coal-mining events were removed from the catalog (see Mueller et al., 1996). Explosions at NTS and their aftershocks were also removed from the catalog.

Attenuation relations for WUS. These relations are discussed below.

Crustal Events: For spectral response acceleration, three equally-weighted attenuation relations were used: (1) Boore, Joyner, and Fumal (BJF; 1993, 1994a) with later modifications to differentiate thrust and strike-slip faulting (Boore et al., 1994b) and (2) Sadigh et al. (1993). For (1) ground motions were calculated for a site with average shear-wave velocity of 760 m/sec in the top 30m, using the relations between shear-wave velocity and site amplification in Boore et al. (1994a). For (2) their "rock" values were used. Joyner (1995) reported velocity profiles compiled by W. Silva and by D. Boore showing that WUS rock sites basically spanned the NEHRP B/C boundary. When calculating ground motions for each fault, the relations appropriate for that fault type (e.g, thrust) were used. All of the relations found higher ground motions for thrust faults compared with strike slip faults.

All calculations included the variability of ground motions. For 1) the sigma values reported in BJF (1994b) were used. For 2) the magnitude-dependent sigmas found in those studies were used.

The distance measure from fault to site varies with the attenuation relation and this was accounted for in the hazard codes (see B.5 for additional detail on distance measures).

Deep events (> 35 km): Most of these events occurred beneath the Puget Sound region, although some were in northwestern California. For these deep events, only one attenuation relation was used – that is, that developed by Geomatrix (1993; with recent modification for depth dependence provided by R. Youngs, written comm., 1996), which is based on empirical data of deep events recorded on rock sites. The relations of Crouse (1991) were used because they were for soil sites. It was found that the ground motions from Geomatrix (1993) are somewhat smaller than those from Crouse (1991), by an amount consistent with soil amplification. These events were placed at a depth of 40 km for calculation of ground motions.

Cascadia subduction zone: For M8.3 events on the subduction zone, two attenuation relations (with equal weights) were used following the lead of Geomatrix (1993): 1) Sadigh et al. (1993) for crustal thrust earthquakes and 2) Geomatrix (1993) for interface earthquakes. For the M9.0 scenario, Sadigh et al. (1993) formulas could not be used since they are invalid over M8.5. Therefore, only Geomatrix (1993) was used. Again the values from Geomatrix (1993) were somewhat smaller than the soil values in Crouse (1991).

ALASKA

The basic procedure, shown in Figure B9, for constructing the Alaska hazard maps is similar to that previously described for the WUS. The maps have been completed and both the maps and documentation (USGS, 1998a, 1998b) have been placed on the USGS internet site (http://geohazards.cr.usgs.gov/eq/); printing of the maps is in progress.

Faults. The hazard from nine faults was used for the maps (Figure B10). Faults were included in the map when an estimated slip rate was available. The seismic hazard associated with faults not explicitly included in the map is captured to a large degree by the smoothed seismicity model. Specific details on the fault parameters are given in USGS., 1997a. All of the faults except one were strike-slip faults.

Recurrence models for faults. As was done for the western U.S., faults were divided into types A and B. The fault treatment was the same as the WUS. Type A faults were the Queen Charlotte, Fairweather offshore, Fairweather onshore, and Transition fault. Type B faults included western Denali, eastern Denali, Totshunda, and Castle Mountain.

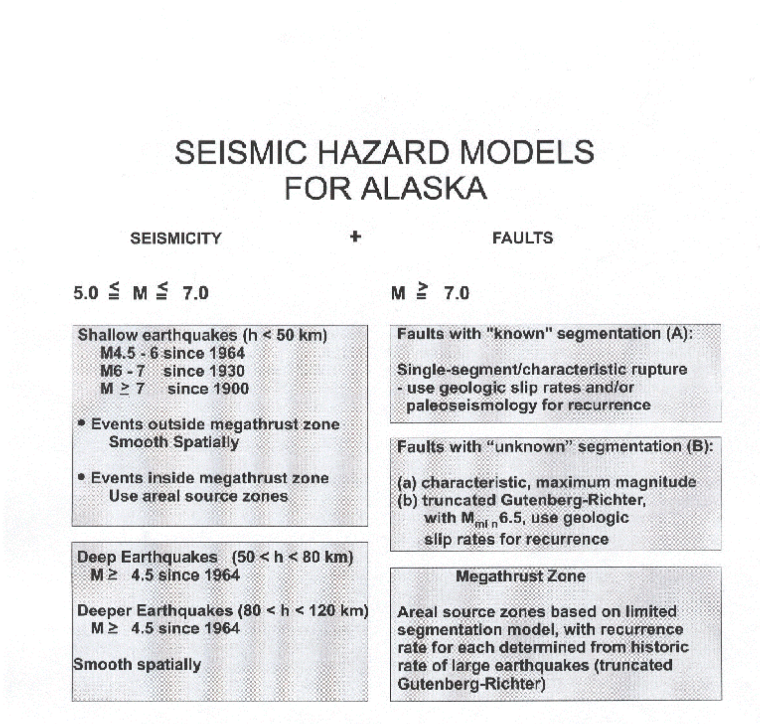

Figure B9 Seismic hazard models for Alaska. Smoothed seismicity models are shown on the left and fault models are shown on the right.

For the type B faults, both characteristic and Gutenberg-Richter (G-R) models of earthquake occurrence were used. These recurrence models were weighted equally. The G-R model accounts for the possibility that a fault is segmented and may rupture only part of its length. It was assumed that the G-R distribution applies from a minimum moment magnitude of 6.5 up to a moment magnitude corresponding to rupture of the entire fault length.

Special case. The Transition fault was treated as a Type A fault even though its segmentation is unknown. Although the rationale for this treatment is documented in USGS, 1998a, it should be pointed out that the parameters, such as segmentation and slip rate, associated with this fault are highly uncertain.

Megathrust. The Alaska-Aleutian megathrust was considered in four parts, shown in Figure B11. Specific rationale for the use of these boundaries is complex and is described in USGS, 1998a.

Alaska catalogs. A new earthquake catalog was built by combining Preliminary Determination of Epicenter, Decade of North American Geology, and International Seismological Centre catalogs with USGS interpretations of catalog reliability. Mueller et al. (1997) describes the processing of the catalogs, the removal of aftershocks, and the assignment of magnitudes.

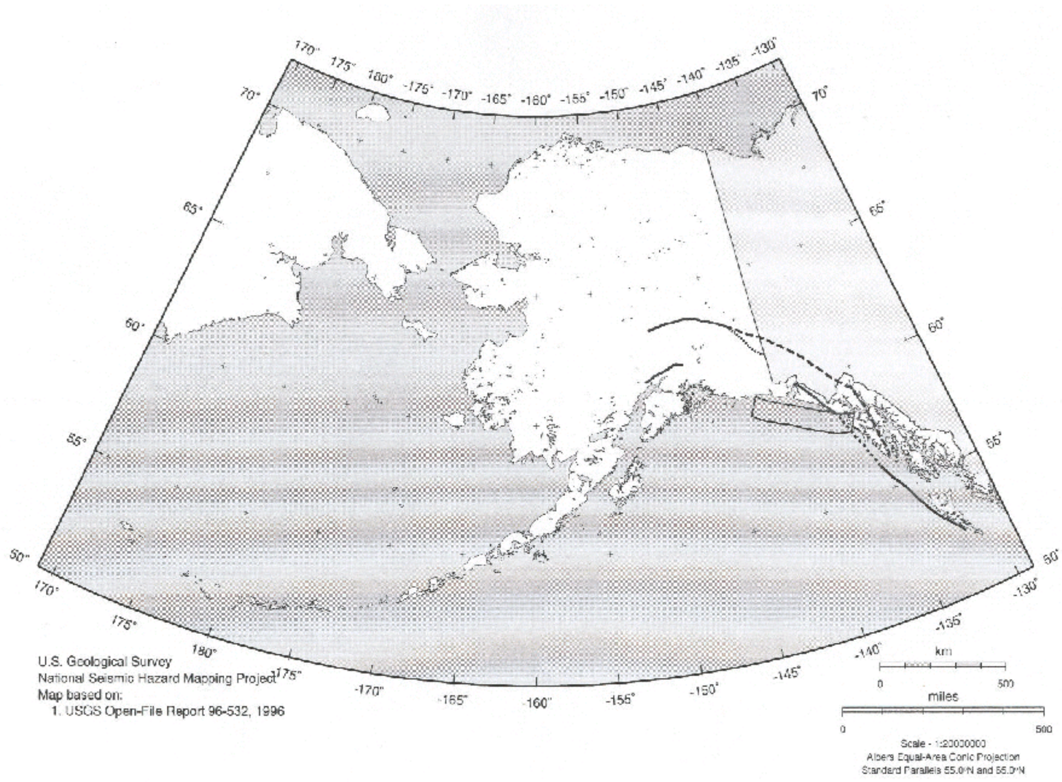

Figure B10 Faults included in the maps. Faults are shown with different line types for clarity. Dipping faults are shown as closed polygons.

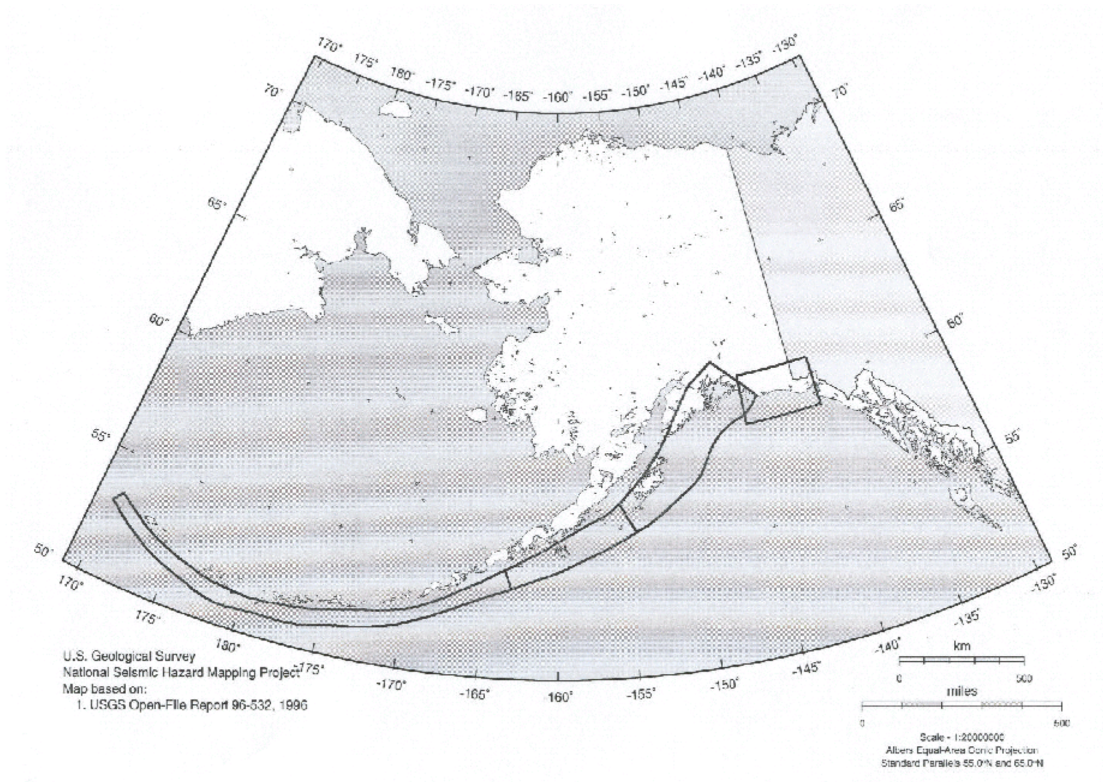

Figure B11 Subduction zones included in the maps

Attenuation relations for Alaska

Crustal Events: For spectral response acceleration, two equally-weighted attenuation relations were used: (1) Boore, Joyner, and Fumal (BJF; 1997) and (2) Sadigh et al. (1997). For (1) ground motions were calculated for a site with average shear-wave velocity of 760 m/sec in the top 30m. For (2) their "rock" values were used. These are recent publication of the attenuations cited for the WUS. The attenuations are the same. When calculating ground motions for each fault, the relations appropriate for that fault type (e.g, strike slip) were used. All calculations included the variability of ground motions.

Deep events (50 - 80 km): For these deep events, only one attenuation relation was used, the intraslab form of Youngs et al. (1997) with a depth fixed at 60 km.

Deeper events (80 - 120 km): For these deeper events, only one attenuation relation was used, the intraslab form of Youngs et al. (1997) with a depth fixed at 90 km.

Megathrust and Transition Fault: Only one attenuation relation was used, the interslab form of Youngs et al. (1997). It should be noted that the use of this attenuation for the Transition fault resulted in lower ground motions than would have been obtained using the crustal attenuation equations.

PROBABILISTIC MAPS

Two of the probabilistic maps were key to the decisions made by the SDPG for developing the maximum considered earthquake ground motion maps. These are the 0.2 sec and 1.0 sec spectral response maps for a 2 percent probability of exceedance in 50 years. These are shown in Figures B12 and B13 respectively. The way in which these maps were used is described in the following sections.

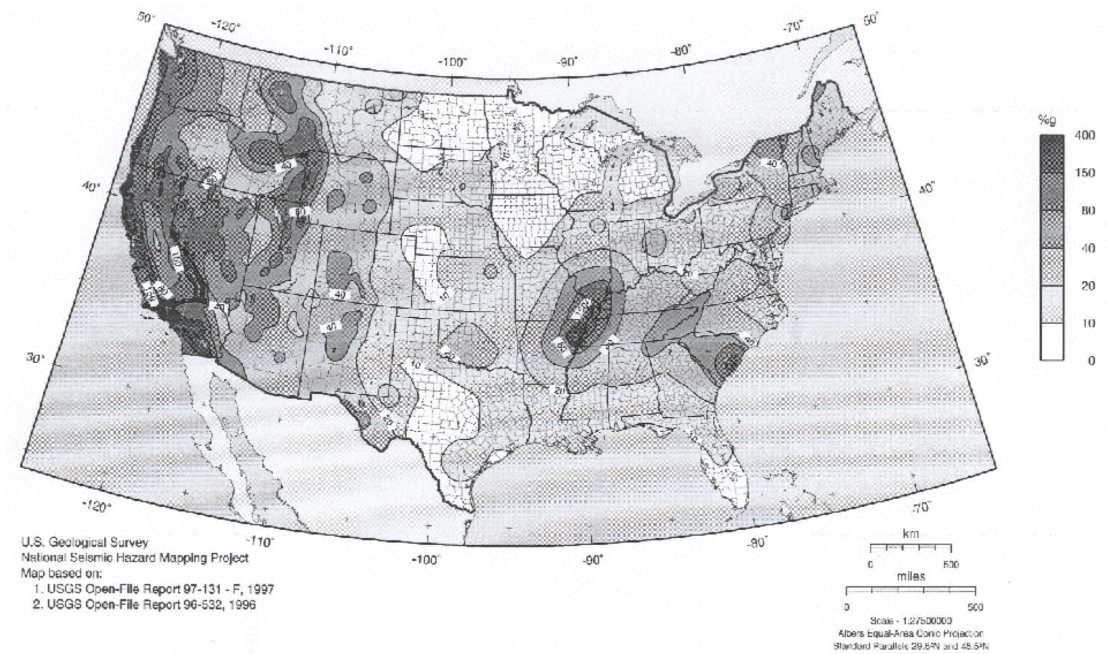

Figure B12 Probabilistic map of 0.2 sec spectral response acceleration with a 2% probability of exceedance in 50 years. The reference site material has a shear wave velocity of 750 m/sec.

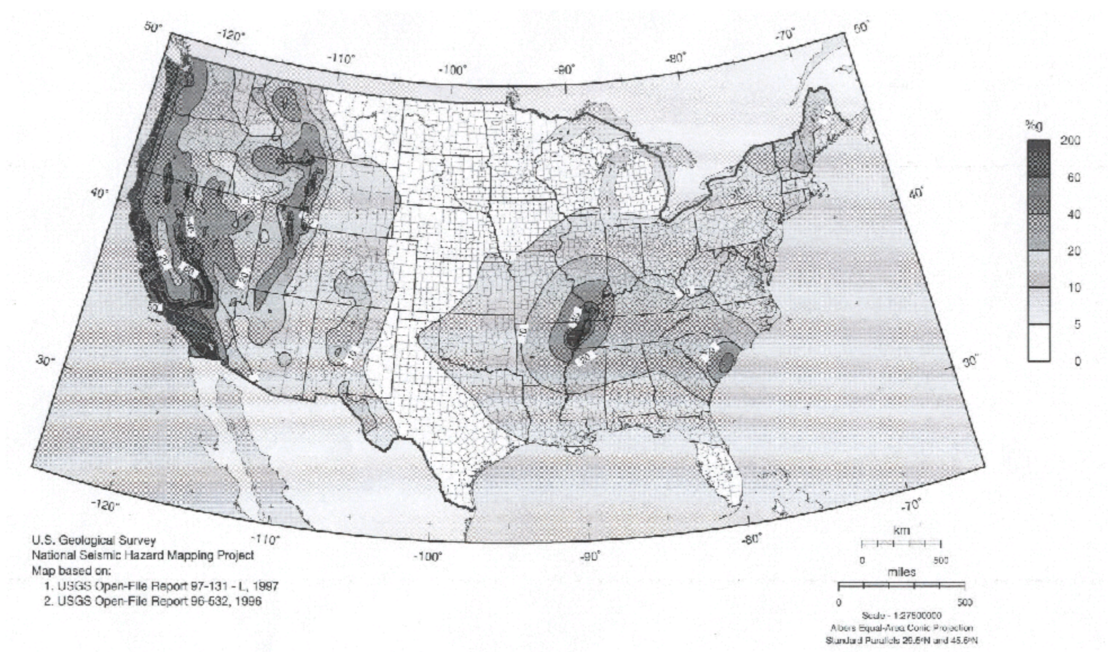

Figure B13 Probabilistic map of 1.0 sec spectral response acceleration with a 2% probability of exceedance in 50 years. The reference site material has a shear wave velocity of 750 m/sec.

DEVELOPMENT OF NEHRP MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION MAPS

The maximum considered earthquake spectral acceleration maps were derived from the 2 percent in 50 year probabilistic maps shown simplified as Figures B12 and B13 (also see Frankel et al, 1997), discussed above, with the application of the SDPG rules also described previously. Additional detail in applying the rules is described in this section. The 0.2 sec map is used for illustration purposes. The same procedures and similar comments apply for the 1.0 sec map.

One of the essential features of the SDPG rules was that the recommendations, when applied by others, would result in the same maps. This procedures allows the use of engineering judgment to be used in developing the maps, as long as those judgments are explicitly stated. This approach will simplify modification of the recommendations as knowledge improves.

It should be noted that although the maps are termed maximum considered earthquake ground motion maps. These maps are not for a single earthquake. The maps include probabilistic effects which consider all possible earthquakes up to the plateau level. Above the plateau level, the contours are included for the deterministic earthquake on each fault (unless the deterministic value is higher than the probabilistic values).

Deterministic contours. The deterministic contours, when included, are computed using the same attenuation functions used in the probabilistic analysis. However, the deterministic values are not used unless they are less than the probabilistic values. After study of those areas where the plateau was reached, the only areas where the deterministic values were less than the probabilistic values were located in California and along the subduction zone region of Washington and Oregon. Further study indicated that those areas with values in excess of the plateau were located in California. The appropriate attenuation for this area were the Boore-Joyner-Fumal attenuation (1993, 1994) and the Sadigh et al. (1993) attenuation.

The form of these attenuations and the distance measures used have an effect on the shape of these deterministic contours. Accordingly, they are discussed below. The Boore-Joyner-Fumal equation is:

$$logY = b_{ss}G_{ss} + b_{RS}G_{RS} + b_2(M - 6)2 + b_4r + b_5log(r) + b_v(logV_s + logV_a)$$

where:

Y	=	ground motion parameter
M	=	earthquake magnitude
b_{SS}, b_{RS}	=	coefficients for strike-slip and reverse-slip faults, determined by regression and different for each ground motion parameter
G_{SS}	=	1.0 for strike-slip fault, otherwise zero
G_{RS}	=	1.0 for reverse-slip fault, otherwise zero
b_2, b_3, b_4, b_5	=	coefficients determined by regression, different for each spectral acceleration
b_V	=	coefficient determined by regression, different for each spectral acceleration
V_A	=	coefficient determined by regression, different for each spectral acceleration
V_S	=	shear wave velocity for different site category
r	=	$(d^2 + h^2)^{½}$
d	=	closest horizontal distance from the site of interest to the surface projection of the rupture surface, see Figure B14
h	=	fictitious depth determined by regression, different for each ground motion parameter

Coefficients determined by regression are tabulated in the reports describing the attenuation equation.

The Sadigh et al. equation is:

$$\ln Y(T) = F\{C_1 + C_2M + C_3(8.5 - M)^{2.5} + C_4\ln[D + \exp(C_5 + C_6M)] + C_y\ln(D + 2)\}$$

where:

Y	=	spectral response acceleration at period T
M	=	earthquake magnitude
C_1, C_2, C3....C_7	=	coefficients determined by regression, different for each ground motion parameter
D	=	closest distance to the fault rupture surface, see Figure B14
F	=	Factor for fault type, 1.0 for strike-slip faults, 1.2 for reverse/thrust faulting, 1.09 for oblique faults

The distance measures are shown in Figure B14 and are discussed in more detail below.

The computation of spectral response (or any ground motion parameter) is a relatively simple matter for a specific site (or specific distance from a fault) but can become complex when preparing contours since it is difficult to calculate the specific distance at which a particular ground motion occurs. This is due to the complexity of the two attenuation functions and the need to combine their results. Since the attenuation functions were weighted equally, each contributes equally to the ground motion at a site. Deterministic contours were determined by preparing attenuation tables, that is the spectral response was computed at various distances from the fault or the fault ends for each earthquake magnitude. Contours for specific values were then drawn by selecting the table for the appropriate magnitude and determining, using interpolation, the distance from the fault for a given spectral acceleration. This procedure required, as a minimum, one attenuation table for each fault. Depending on the fault geometry, more than one table was needed. In order to illustrate this the strike-slip fault is discussed first, followed by a discussion of dipping faults.

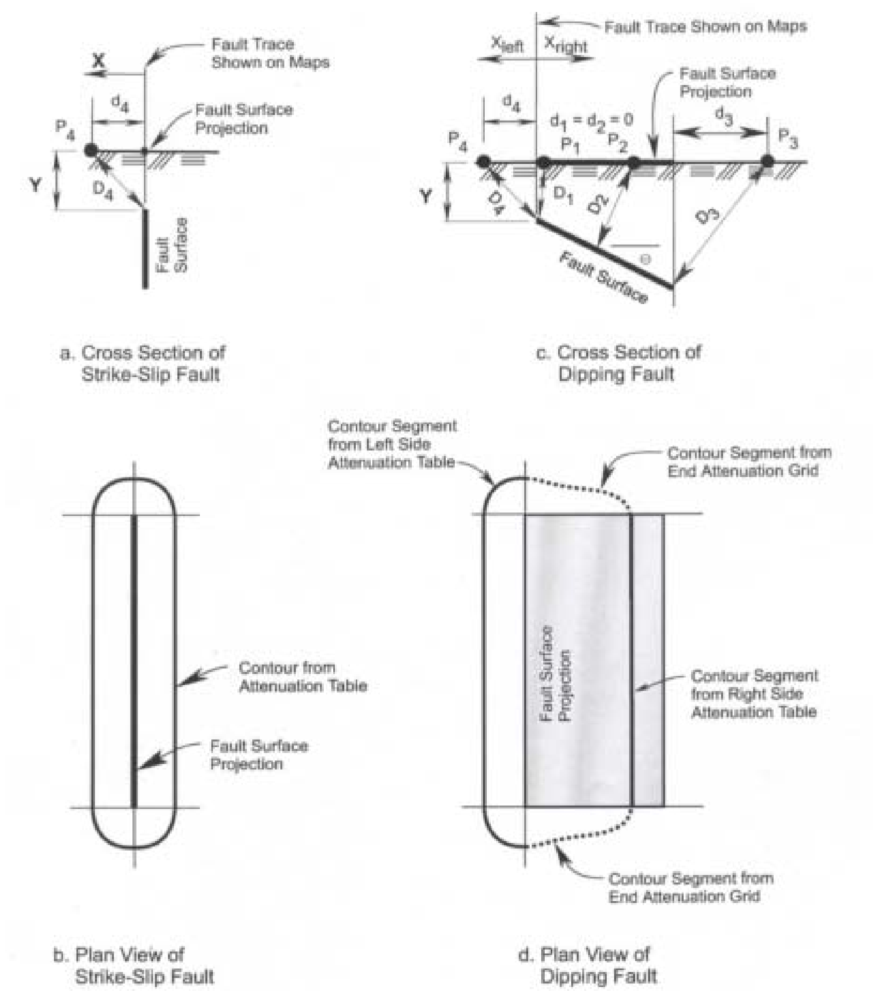

Figure B14 Measures of distance for strike-slip and dipping faults. A cross section of strike-slip fault is shown in figure (a) and the shape of a typical deterministic contour is shown in figure (b). A dipping fault is shown in figure (c) and the shape of a typical deterministic contour is shown in figure (d).

Strike-slip faults: The strike-slip fault, shown in Figure B14a, b is the simplest introduction to application of the SDPG rules. The distance measures are shown for each attenuation function in Figure B14a. The Boore-Joyner-Fumal equation uses the distance, d_4. The term r in equation includes d_4 and the fictitious depth h. Since h is not zero, $r > d_4$, even if the term y in Figure B14a is zero. The Sadigh et al. equation measures the distance, D, as the closest distance to the rupture surface. In this case to the top of the rupture. If the depth y is zero, then $d_4 = D_4$.

It makes little difference in the computations if the fault rupture plane begins at the surface or at some distance below the surface. For the strike-slip fault the contour for a particular spectral acceleration is a constant distant from the fault and the contour is as shown in Figure B14b. One attenuation table (including the effects of both attenuation equations) can be used for either side of the fault and at the fault ends.

Dipping faults: The dipping fault, shown in Figures B14c and d, is the most complex case for preparing deterministic contours. The distance measures are shown for each attenuation function in Figure B14c. As before, it is a simple matter to compute the spectral values at a specific site, but not as simple to compute the distance at which a specific spectral acceleration occurs. This is particularly true at the end of the fault.

On the left side of the fault shown in Figure B14c, an attenuation table is prepared, much as in the case of the strike-slip fault. This table may also be used to determine the contour around a portion of the fault end as shown in Figure B14d. In this case it is simply one-quarter of a circle.

A separate attenuation table must be prepared for the right side of the fault as shown in Figure B14d. Since d or D is measured differently, depending on location x, calculations must keep track of whether or not the location is inside or outside of the surface fault projection. Note that the term d is zero when the location x falls within the surface projection, but the fictitious depth h is not. Outside the fault projection, the distance d is measured from the edge of the projection. The distance D is calculated differently, as illustrated in Figure B14c, depending on location but it is always the closest distance to the fault rupture surface.

At the ends of the fault, an attenuation grid was prepared to determine the contour shape shown dotted in Figure B14d. The contour in this area was digitized using the gridded values and combined with the remainder of the contour determined from the left and right attenuation tables. This need for digitizing a portion of the contour greatly increased the time required to prepare each of the contours for dipping faults. In short, each dipping fault required two attenuation tables and an attenuation grid to prepare each deterministic contour. Thus preparation of each contour is far more time-consuming than preparing a contour for a strike-slip fault. Each contour is unsymmetrical around the fault, the amount of asymmetry depends on the angle of dip.

It can be argued that the knowledge of fault locations and geometry does not warrant this level of effort. However, it was considered necessary in order to follow the concept of repeatability in preparing the maps.

Combining deterministic contours: Where two or more faults are nearby, as in Figure B15a, the deterministic contours were merged (depending on amplitudes) as shown in Figure B15b. The merging resulted in the sharp "corners" shown in the figure. Although it can be argued that these intersections should be smoothed, it was believed that maintaining the shape reflected the decision to use deterministic contours.

Combining deterministic and probabilistic contours. The SDPG decision to use a combination of deterministic and probabilistic contours, although simple in principle, led to number of problems in preparing the contour maps.

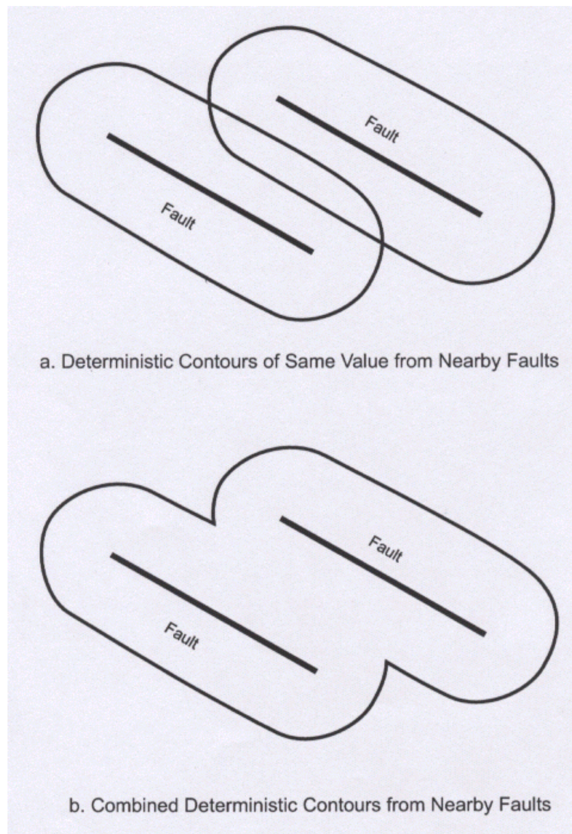

Figure B15 Procedure for combining deterministic contours from nearby faults

Figure B16a, b for a single strike-slip fault illustrates the concept originally envisioned for combining the deterministic and probabilistic contours. After combining the two sets of contours shown in Figure B16a, the maximum considered earthquake contours would be as shown in Figure B16b.

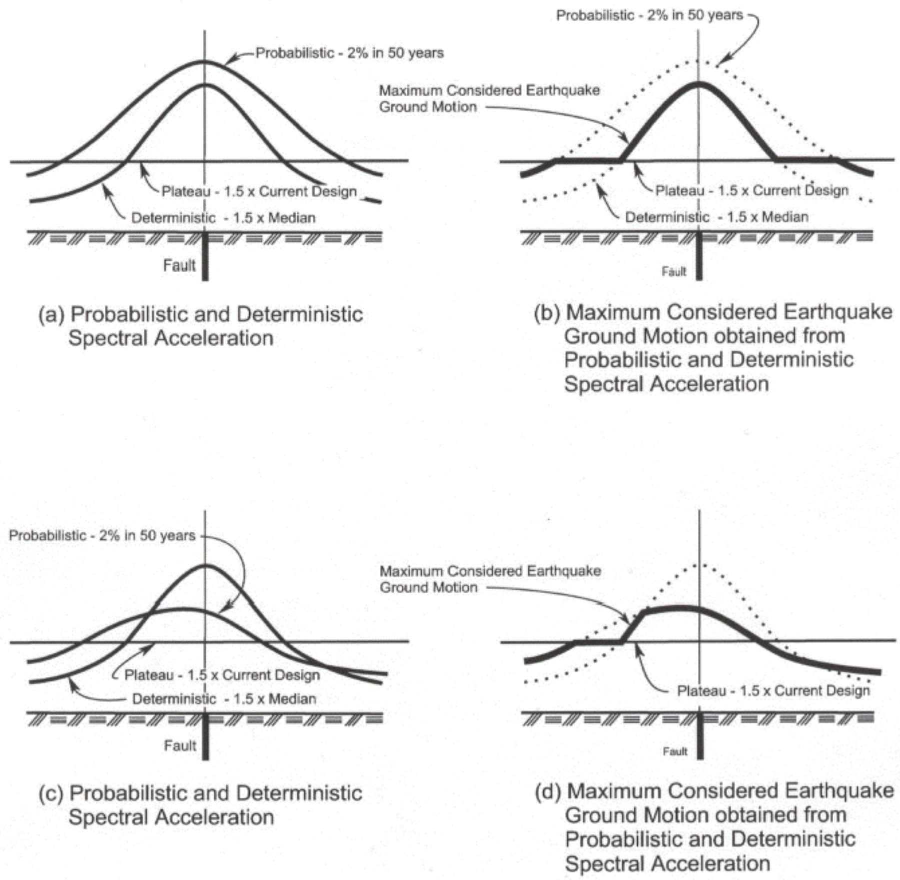

Figure B16 Procedure for obtaining maximum considered earthquake ground motion

In application the situation is more complex, there is frequently more than one fault, with different magnitudes, different return times, different fault geometry, and different locations with respect to each other. Examples are shown in Figures 17 and 18 which will be discussed later. The effect of the variables is illustrated in Figure B16 c and d. The deterministic curve is shown for a single fault with a return time much larger than that of the map. The deterministic spectral acceleration is much larger than the spectral acceleration resulting from historical seismicity. The probabilistic curve is not necessarily symmetrical to the fault. The resulting maximum considered earthquake curve shown in Figure B16d is a complex mix of the probabilistic and deterministic curves. There is not always a plateau and the curve is not necessarily symmetrical to the fault, even for a strike-slip fault. Simply stated, the probabilistic curve considers other sources such as historical seismicity and other faults as well as time. The deterministic curve does not consider other sources for this simple example and does not consider time.

The only areas of the United States that have deterministic contours are in California, along the Pacific coast through Oregon and Washington, and in Alaska. At first review it can be seen that there are several other areas that have contours in excess of the plateau but do not have plateaus. In these areas (e.g., New Madrid), the deterministic values exceed the probabilistic ones and thus were not used.

There were several instances where application of the SDPG rules produced results that appear counterintuitive and in other instance produced results that were edited. Two examples from southern

California are discussed below. Each example is illustrated with a three-part figure. Part (a) shows both probabilistic contours (dashed) and deterministic contours (solid) for each fault which is also shown. Part (b) shows the maximum considered earthquake results produced by following the SDPG rules. Part (c) shows how part (b) was edited for the final map.

Example 1: The first example in Figure B17 illustrates the occurrence of gaps in the deterministic contours around a fault and the halt of a deterministic contour before the end of a fault. When the probabilistic contours and deterministic contours shown in Figure B17a are combined, a gap in the deterministic contours occurs in the vicinity of 34.6° and 118.8°. Similarly the deterministic contours stop prior to the end of the fault around 34.65° and 119.4°. Both of these are shown in Figure B17b.

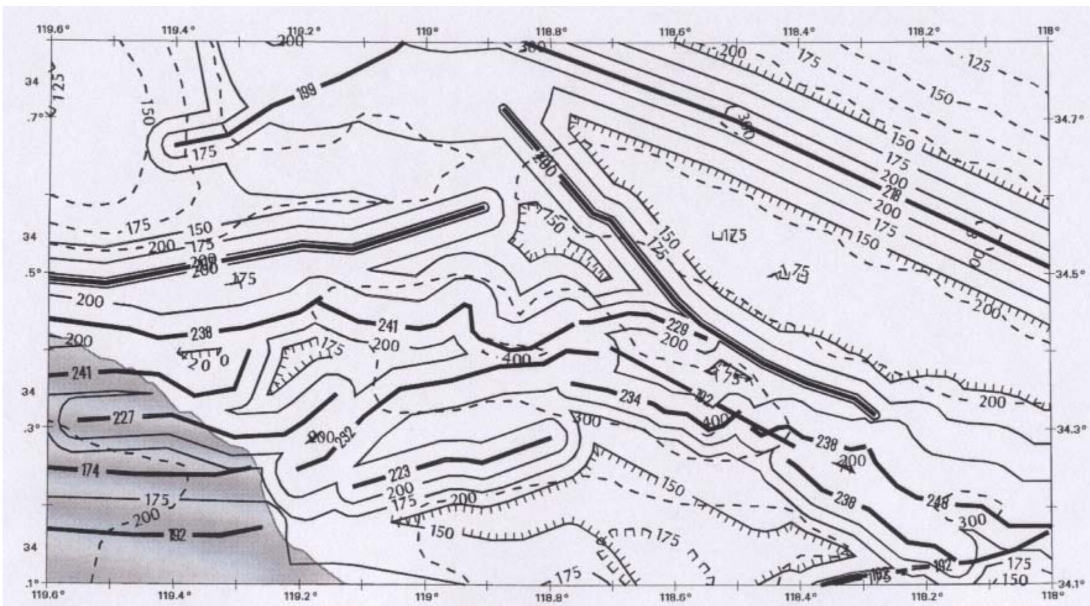

Figure B17a Combining contours - Example 1. Both probabilistic and deterministic contours are shown. Probabilistic contours are shown dashed.

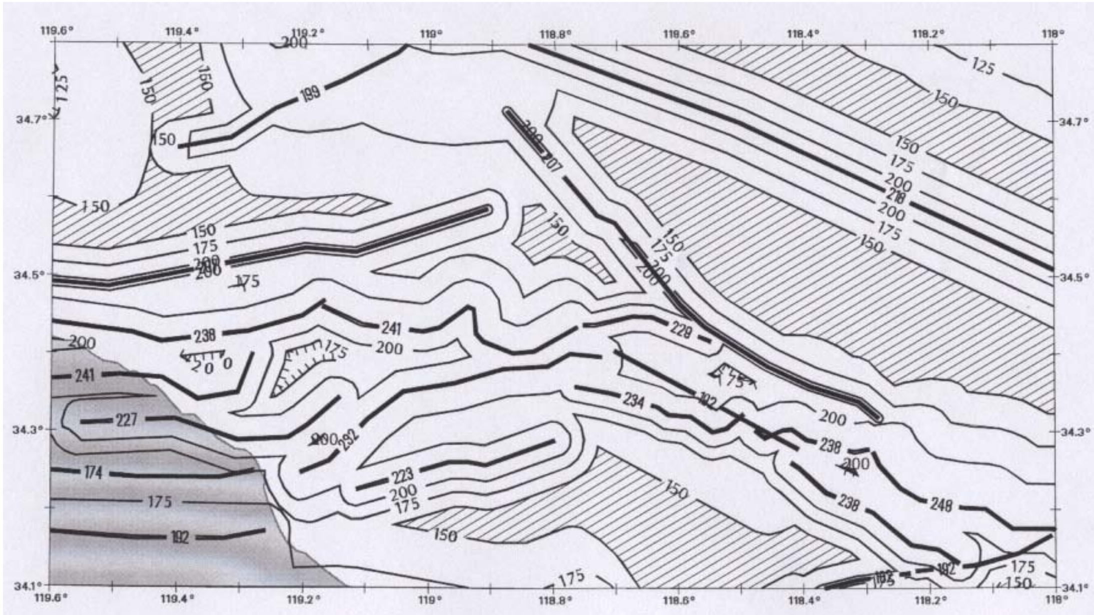

Figure B17b Combining contours - Example 1. Both probabilistic contours are merged using strict interpretation of committee rules.

After study, it is clear that the SDPG rules results in a repeatable, but unusual, set of contours. The result does not go along with the concept of accounting for near fault effects with the deterministic contours. Because of this undesirable effect, the contours were hand edited to restore the gaps and produce the result in Figure B17c.

All occurrences similar to this were edited to modify the contours so that the deterministic contours did not have abrupt breaks or stops before the ends of the fault.

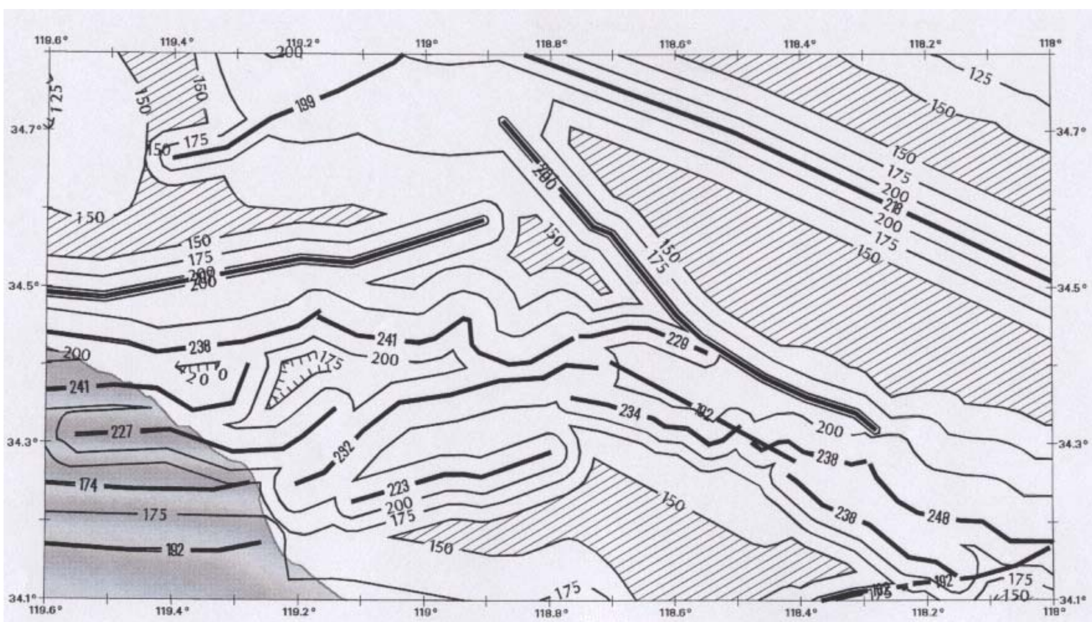

Figure B17c Combining contours - Example 1. Probabilistic contours are merged with deterministic contours using strict interpretation of committee rules with subsequent editing.

Example 2: The second example in Figure B18 illustrates the occurrence of many faults at different orientations to each other and with different return times. Merging of the complex set of contours is shown in Figure B18b. The contours are greatly simplified. Some small plateaus are shown along the 150 percent contour, as is a gap along one of the faults around 34.0° and 116.35°. The gap was edited as in example 1. The small plateaus were edited out using the judgment that their presence was inconsequential (less than a few percent effect on the maps) and unnecessarily complicated an already complicated map.

Another problem created was that some of the faults have portions of the fault, with a specific acceleration value, in areas where the contours are less than the fault value. An example occurs with the fault labeled 248 in the vicinity of 34.4° and 117.2°. A footnote was added to the maximum considered earthquake maps to the effect that the fault value was only to be used in areas where it exceeded the surrounding contours. Although other approaches are possible, such as showing the unused portion of the fault dashed, the full length of the faults are shown solid in the maps.

As shown in Figure B18b, a sawtooth contour around 34.15° and 116.3° results from application of committee rules. Although this appears to be a candidate for smoothing, it was not done as shown in Figure B18c. Once again there are several possible ways to smooth but it was not done in the interest of repeatability.

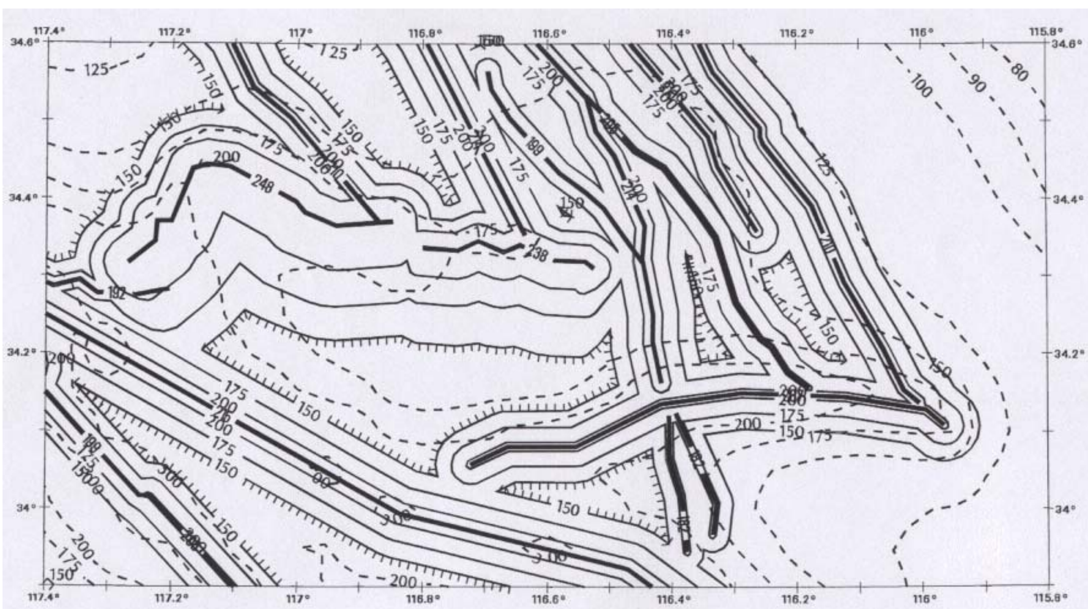

Figure B18a Combining contours - Example 2. Both probabilistic and deterministic contours are shown. Probabilistic contours are shown dotted.

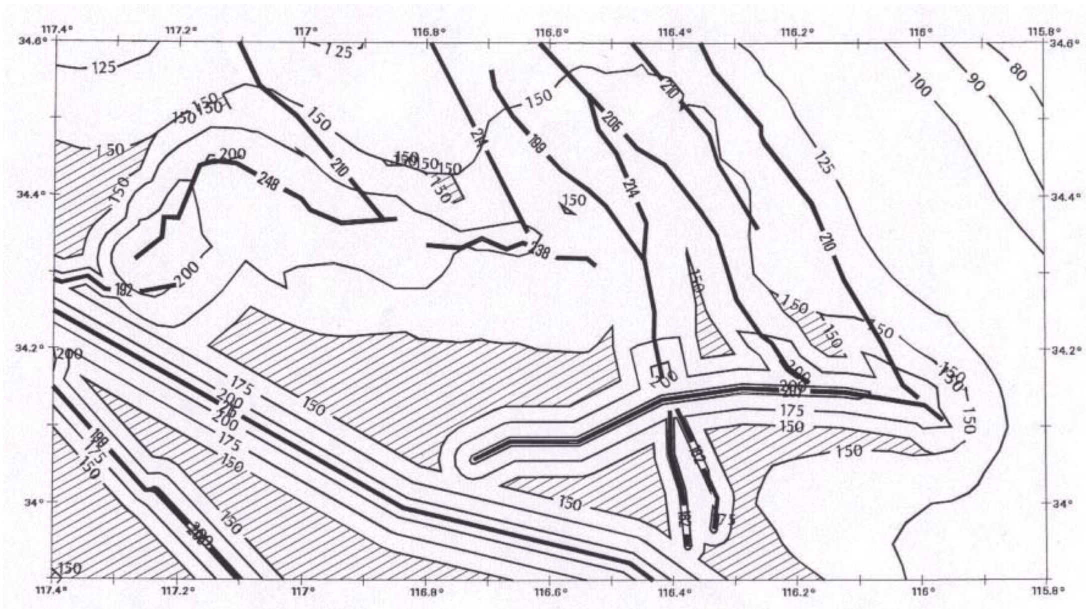

Figure B18b Combining contours - Example 2. Probabilistic contours are merged using strict interpretation of committee rules.

Probability level. The maximum considered earthquake spectral acceleration maps use the 2 percent in 50 maps as a base; however, the values obtained from the maps are multiplied by 2/3 for use in the design equation. This implicitly results in a different probability being used in different areas of the United States. The hazard curves shown in Figure B2 are normalized to the 2 percent in 50 year value in Figure B19. This figure shows that the slope of the hazard curve varies in different areas of the United States. In general, the curves are steeper for CEUS cities than for WUS cities with the WUS curves beginning to flatten out earlier than the CEUS cities. Typical curves for a CEUS and WUS city are shown in Figure B20. This figure shows than when the 2/3 factor is applied, probabilistic values a for WUS location are close to a 10 percent in 50 year value and probabilities for CEUS locations reflect a lower probability.

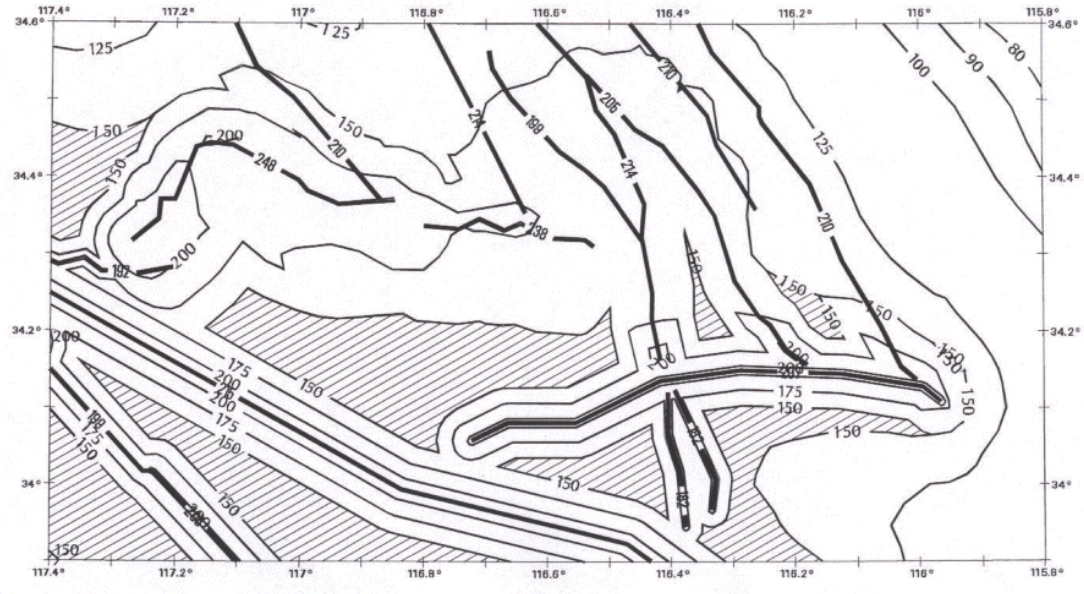

Figure B18c Combining contours - Example 2. Probabilistic contours are merged with deterministic contours using strict interpretation of committee rules with subsequent editing.

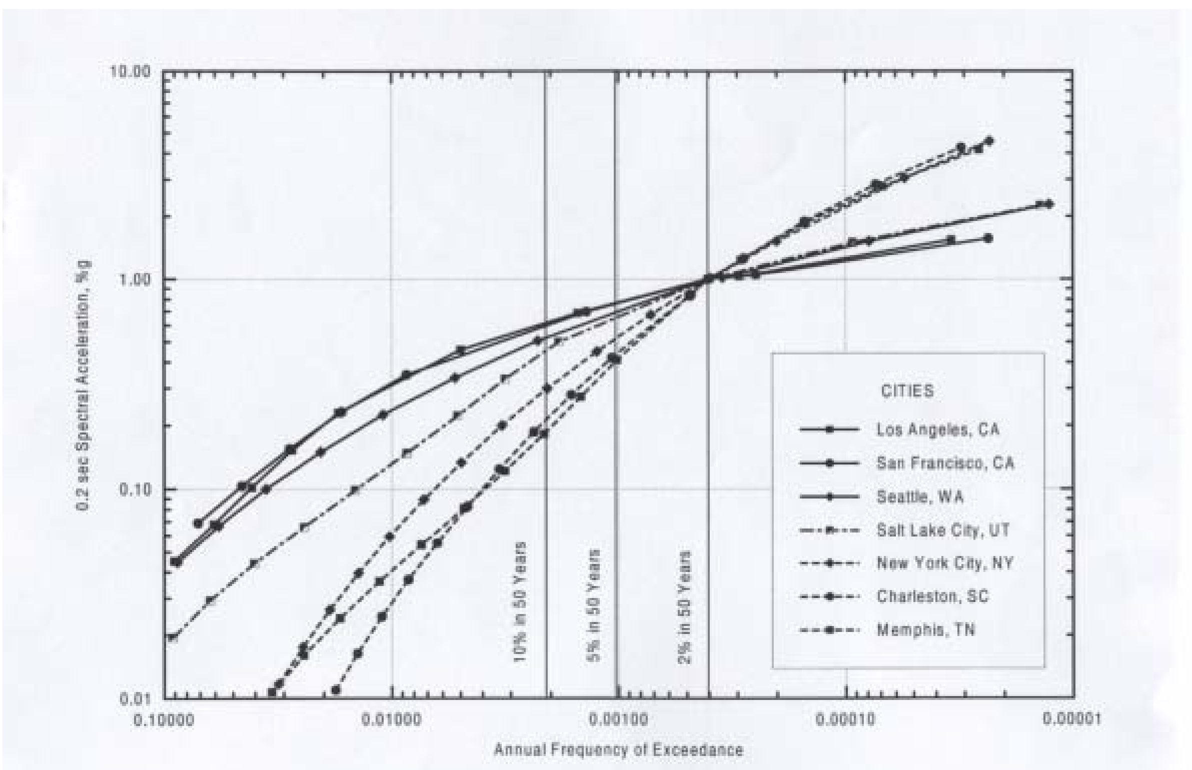

Figure B19 Hazard curves for selected cities. The curves are normalized to 2% in 50 years.

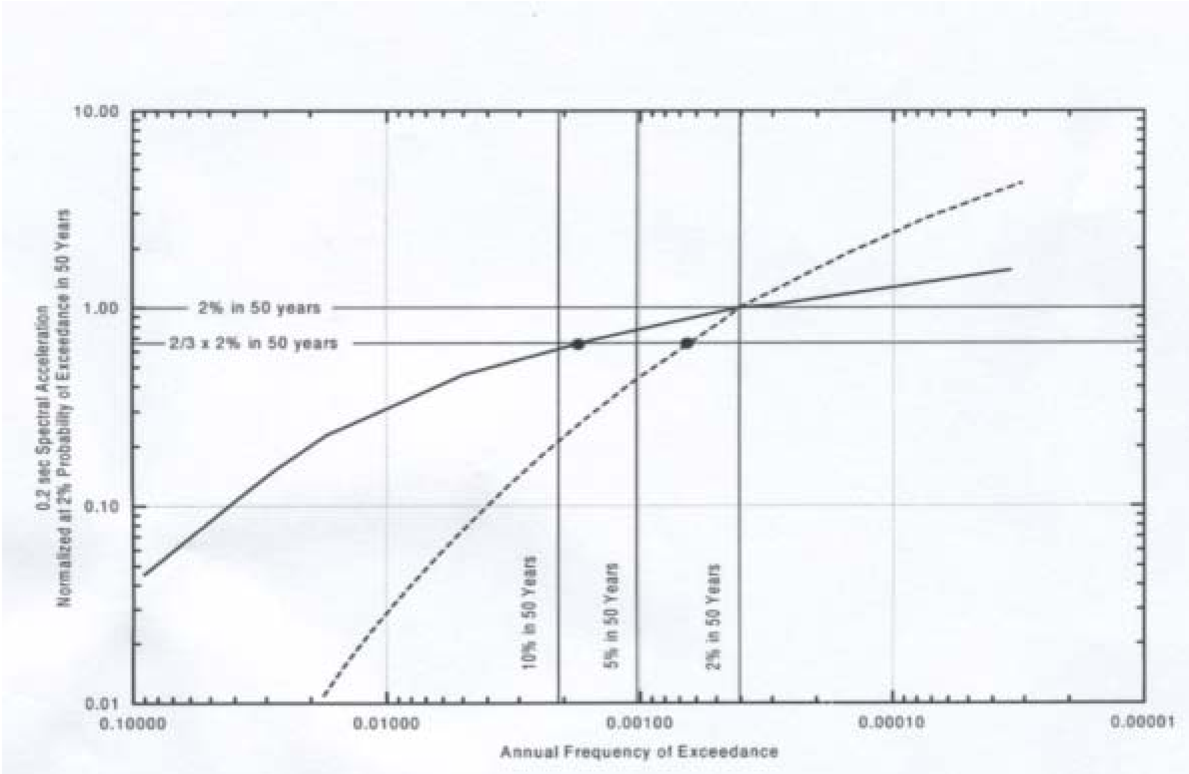

Figure B20 Effect on the probability level of multiplying the spectral acceleration by 2/3

Interpolation. Linear interpolation between contours is permitted using the maximum considered earthquake maps. To facilitate interpolation, spot values have been provided inside closed contours of increasing or decreasing values of the design parameter. Additional spot values have been provided where linear interpolation would be difficult. Values have also been provided along faults in the deterministic areas to aid in interpolation.

Hawaii. The Hawaii State Earthquake Advisory Board (HSEAB), in its ballot on the 1997 *Provisions*, proposed different maps from those included in the original BSSC ballot. The HSEAB's comments were based in part on recent work done to propose changes in seismic zonation for the 1994 and 1997 *Uniform Building Code*. The HSEAB also was concerned that in early 1998 the USGS would be completing maps that would be more up to date then those included in the original BSSC ballot. Essentially, the HSEAB's recommendation was that the maps it submitted or the new USGS maps should be used for Hawaii. The USGS maps were completed in March 1998 and were reviewed by the HSEAB, including proposals for incorporation of deterministic contours where the ground motions exceed the plateau levels described previously. The maps were revised in response to review comments and the modified design maps are included as part of the *Provisions*.

Briefly, the probabilistic maps were prepared using a USGS methodology similar to that used for the western United States (Klein et. al.). Two attenuation functions were used: Sadigh as described earlier and Munson and Thurber, which incorporates Hawaii data. The Hawaii contour maps (*Provisions* Map 10) are probabilistic except for two areas on the island of Hawaii. The two areas (outlined by the heavy border on Map 10) are located on the western and southeastern portion of the island. The two areas are defined by horizontal rupture planes at a 9 km depth. Within these zones, the spectral accelerations are constant. The western zone uses a magnitude 7.0 event while the southwestern zones uses a magnitude 8.2 event. The deterministic values inside the zone and for the contours were calculated as described in earlier sections.

Additional maximum considered earthquake ground motion maps. Maps for Puerto Rico and the U.S. Virgin islands were prepared using the USGS methodology described previously with modifications and attenuations appropriate for the region as described by Mueller, et. al. The two maximum considered earthquake spectral acceleration maps for the region are entirely probabilistic since values did not exceed the thresholds requiring incorporation of deterministic values. Although new probabilistic maps were not available for Guam and Tutuila, maximum considered earthquake maps were required for use by the *Provisions*. Maximum considered earthquake spectral response maps for these areas were prepared as follows.

Maps for Guam and Tutuila were prepared using the 1994 NEHRP maps. These were for approximately 10 percent probability of exceedance in 50 years. The ratio of PGA for 2 percent in 50 years to 10 percent in 50 years for the new USGS maps is about two. Accordingly maps for these areas were converted to 2 percent in 50 year maps by multiplying by two. These maps were then converted to spectral maps by using the factors described below.

A study of the ratios of the 0.2 sec and 1.0 sec spectral responses to PGA was done. Although approximate, the ratios were about 2.25 to 2.5 for the 0.2 sec spectral acceleration and about 1.0 for the 1.0 sec response. Thus PGA for the above regions was converted to spectral acceleration by multiplying PGA by 2.5 for the 0.2 sec response and by 1.0 for the 1.0 sec response. It should be noted that the multiplier for the 1.0 sec response varied over a wider range than the 0.2 sec response multiplier. It should be used cautiously.

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THE COUNCIL: PROJECTS AND ACTIVITIES

The Building Seismic Safety Council (BSSC) was established in 1979 under the auspices of the National Institute of Building Sciences as an entirely new type of instrument for dealing with the complex regulatory, technical, social, and economic issues involved in developing and promulgating building earthquake risk mitigation regulatory provisions that are national in scope. By bringing together in the BSSC all of the needed expertise and all relevant public and private interests, it was believed that issues related to the seismic safety of the built environment could be resolved and jurisdictional problems overcome through authoritative guidance and assistance backed by a broad consensus.

The BSSC is an independent, voluntary membership body representing a wide variety of building community interests. Its fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings. To fulfill its purpose, the BSSC:

- Promotes the development of seismic safety provisions suitable for use throughout the United States;
- Recommends, encourages, and promotes the adoption of appropriate seismic safety provisions in voluntary standards and model codes;
- Assesses progress in the implementation of such provisions by federal, state, and local regulatory and construction agencies;
- Identifies opportunities for improving seismic safety regulations and practices and encourages public and private organizations to effect such improvements;
- Promotes the development of training and educational courses and materials for use by design professionals, builders, building regulatory officials, elected officials, industry representatives, other members of the building community, and the public;
- Advises government bodies on their programs of research, development, and implementation; and
- Periodically reviews and evaluates research findings, practices, and experience and makes recommendations for incorporation into seismic design practices.

The BSSC's area of interest encompasses all building types, structures, and related facilities and includes explicit consideration and assessment of the social, technical, administrative, political, legal, and economic implications of its deliberations and recommendations. The BSSC believes that the achievement of its purpose is a concern shared by all in the public and private sectors; therefore, its activities are structured to provide all interested entities (i.e., government bodies at all levels, voluntary organizations, business, industry, the design profession, the construction industry, the research community, and the general public) with the opportunity to participate. The BSSC also believes that the regional and local differences in the nature and magnitude of potentially hazardous earthquake events require a flexible approach to seismic safety that allows for consideration of the relative risk, resources, and capabilities of each community. The BSSC is committed to continued technical improvement of seismic design provisions, assessment of advances in engineering knowledge and design experience, and evaluation of earthquake impacts. It recognizes that appropriate earthquake hazard risk reduction measures and initiatives should be adopted by existing organizations and institutions and incorporated, whenever possible, into their legislation, regulations, practices, rules, codes, relief procedures, and loan requirements so that these measures and initiatives become an integral part of established activities, not additional burdens. Thus, the BSSC itself assumes no standards-making role; rather, it advocates that code- and standards-formulation organizations consider the BSSC's recommendations for inclusion in their documents and standards.

IMPROVING THE SEISMIC SAFETY OF NEW BUILDINGS

The BSSC program directed toward improving the seismic safety of new buildings has been conducted with funding from the Federal Emergency Management Agency (FEMA). It is structured to create and maintain authoritative, technically sound, up-to-date resource documents that can be used by the voluntary standards and model code organizations, the building community, the research community, and the public as the foundation for improved seismic safety design provisions.

The BSSC program began with initiatives taken by the National Science Foundation (NSF). Under an agreement with the National Institute of Standards and Technology (NIST; formerly the National Bureau of Standards), *Tentative Provisions for the Development of Seismic Regulations for Buildings* (referred to here as the *Tentative Provisions*) was prepared by the Applied Technology Council (ATC). The ATC document was described as the product of a "cooperative effort with the design professions, building code interests, and the research community" intended to "...present, in one comprehensive document, the current state of knowledge in the fields of engineering seismology and engineering practice as it pertains to seismic design and construction of buildings." The document, however, included many innovations, and the ATC explained that a careful assessment was needed.

Following the issuance of the *Tentative Provisions* in 1978, NIST released a technical note calling for " . . . systematic analysis of the logic and internal consistency of [the *Tentative Provisions*]" and developed a plan for assessing and implementing seismic design provisions for buildings. This plan called for a thorough review of the *Tentative Provisions* by all interested organizations; the conduct of trial designs to establish the technical validity of the new provisions and to assess their economic impact; the establishment of a mechanism to encourage consideration and adoption of the new provisions by organizations promulgating national standards and model codes; and educational, technical, and administrative assistance to facilitate implementation and enforcement.

During this same period, other significant events occurred. In October 1977, Congress passed the *Earthquake Hazards Reduction Act of 1977* (P.L. 95-124) and, in June 1978, the National Earthquake Hazards Reduction Program (NEHRP) was created. Further, FEMA was established as an independent agency to coordinate all emergency management functions at the federal level. Thus, the future disposition of the *Tentative Provisions* and the 1978 NIST plan shifted to FEMA. The emergence of FEMA as the agency responsible for implementation of P.L. 95-124 (as amended) and the NEHRP also required the creation of a mechanism for obtaining broad public and private consensus on both recommended improved building design and construction regulatory provisions and the means to be used in their promulgation. Following a series of meetings between representatives of the original participants in the NSF-sponsored project on seismic design provisions, FEMA, the American Society of Civil Engineers and the National Institute of Building Sciences (NIBS), the concept of the Building Seismic Safety Council was born. As the concept began to take form, progressively wider public and private participation was sought, culminating in a broadly representative organizing meeting in the spring of 1979, at which time a charter and organizational rules and procedures were thoroughly debated and agreed upon.

The BSSC provided the mechanism or forum needed to encourage consideration and adoption of the new provisions by the relevant organizations. A joint BSSC-NIST committee was formed to conduct the needed review of the *Tentative Provisions*, which resulted in 198 recommendations for changes. Another joint BSSC-NIST committee developed both the criteria by which the needed trial designs could be evaluated and the specific trial design program plan. Subsequently, a BSSC-NIST Trial Design Overview Committee was created to revise the trial design plan to accommodate a multiphased effort and to refine the *Tentative Provisions*, to the extent practicable, to reflect the recommendations generated during the earlier review.

Trial Designs

Initially, the BSSC trial design effort was to be conducted in two phases and was to include trial designs for 100 new buildings in 11 major cities, but financial limitations required that the program be scaled down. Ultimately, 17 design firms were retained to prepare trial designs for 46 new buildings in 4 cities with medium to high seismic risk (10 in Los Angeles, 4 in Seattle, 6 in Memphis, 6 in Phoenix) and in 5 cities with medium to low seismic risk (3 in Charleston, South Carolina, 4 in Chicago, 3 in Ft. Worth, 7 in New York, and 3 in St. Louis). Alternative designs for six of these buildings also were included.
The firms participating in the trial design program were: ABAM Engineers, Inc.; Alfred Benesch and Company; Allen and Hoshall; Bruce C. Olsen; Datum/Moore Partnership; Ellers, Oakley, Chester, and Rike, Inc.; Enwright Associates, Inc.; Johnson and Nielsen Associates; Klein and Hoffman, Inc.; Magadini-Alagia Associates; Read Jones Christoffersen, Inc.; Robertson, Fowler, and Associates; S. B. Barnes and Associates; Skilling Ward Rogers Barkshire, Inc.; Theiss Engineers, Inc.; Weidlinger Associates; and Wheeler and Gray.

For each of the 52 designs, a set of general specifications was developed, but the responsible design engineering firms were given latitude to ensure that building design parameters were compatible with local construction practice. The designers were not permitted, however, to change the basic structural type even if an alternative structural type would have cost less than the specified type under the early version of the *Provisions*, and this constraint may have prevented some designers from selecting the most economical system.

Each building was designed twice – once according to the amended *Tentative Provisions* and again according to the prevailing local code for the particular location of the design. In this context, basic structural designs (complete enough to assess the cost of the structural portion of the building), partial structural designs (special studies to test specific parameters, provisions, or objectives), partial nonstructural designs (complete enough to assess the cost of the nonstructural portion of the building), and design/construction cost estimates were developed.

This phase of the BSSC program concluded with publication of a draft version of the recommended provisions, the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, an overview of the *Provisions* refinement and trial design efforts, and the design firms' reports.

The 1985 Edition of the *NEHRP Recommended Provisions*

The draft version represented an interim set of provisions pending their balloting by the BSSC member organizations. The first ballot, conducted in accordance with the BSSC Charter, was organized on a chapter-by-chapter basis. As required by BSSC procedures, the ballot provided for four responses: "yes," "yes with reservations," "no," and "abstain." All "yes with reservations" and "no" votes were to be accompanied by an explanation of the reasons for the vote and the "no" votes were to be accompanied by specific suggestions for change if those changes would change the negative vote to an affirmative.

All comments and explanations received with "yes with reservations" and "no" votes were compiled, and proposals for dealing with them were developed for consideration by the Technical Overview Committee and, subsequently, the BSSC Board of Direction. The draft provisions then were revised to reflect the changes deemed appropriate by the BSSC Board and the revision was submitted to the BSSC membership for balloting again. As a result of this second ballot, virtually the entire provisions document received consensus approval, and a special BSSC Council meeting was held in November 1985 to resolve as many of the remaining issues as possible. The 1985 Edition of the *NEHRP Recommended Provisions* then was transmitted to FEMA for publication in December 1985.

During the next three years, a number of documents were published to support and complement the 1985 *Provisions*. They included a guide to application of the *Provisions* in earthquake-resistant building design, a nontechnical explanation of the *Provisions* for the lay reader, and a handbook for interested members of the building community and others explaining the societal implications of utilizing improved seismic safety provisions and a companion volume of selected readings.

The 1988 Edition

The need for continuing revision of the *Provisions* had been anticipated since the onset of the BSSC program and the effort to update the 1985 Edition for reissuance in 1988 began in January 1986. During the update effort, nine BSSC Technical Committees (TCs) studied issues concerning seismic risk maps, structural design, foundations, concrete, masonry, steel, wood, architectural and mechanical and electrical systems, and regulatory use. The Technical Committees worked under the general direction of a Technical Management Committee (TMC), which was composed of a representative of each TC as well as additional members identified by the BSSC Board to provide balance.

The TCs and TMC worked throughout 1987 to develop specific proposals for changes needed in the 1985 *Provisions*. In December 1987, the Board reviewed these proposals and decided upon a set of 53 for submittal to the BSSC membership for ballot. Approximately half of the proposals reflected new issues while the other half reflected efforts to deal with unresolved 1985 edition issues.

The balloting was conducted on a proposal-by-proposal basis in February-April 1988. Fifty of the proposals on the ballot passed and three failed. All comments and "yes with reservation" and "no" votes received as a result of the ballot were compiled for review by the TMC. Many of the comments could be addressed by making minor editorial adjustments and these were approved by the BSSC Board. Other comments were found to be unpersuasive or in need of further study during the next update cycle (to prepare the 1991 *Provisions*). A number of comments persuaded the TMC and Board that a substantial alteration of some balloted proposals was necessary, and it was decided to submit these matters (11 in all) to the BSSC membership for reballot during June-July 1988. Nine of the eleven reballot proposals passed and two failed.

On the basis of the ballot and reballot results, the 1988 *Provisions* documents were prepared and transmitted to FEMA for publication in August 1988. A report describing the changes made in the 1985 edition and issues in need of attention in the next update cycle also was prepared, and efforts to update the complementary reports published to support the 1985 edition were initiated. Ultimately, the following publications were updated to reflect the 1988 Edition and reissued by FEMA: the *Guide to Application of the Provisions*, the handbook discussing societal implications (which was extensively revised and retitled *Seismic Considerations for Communities at Risk*), and several *Seismic Considerations* handbooks (which are described below).

The 1991 Edition

During the effort to produce the 1991 *Provisions*, a Provisions Update Committee (PUC) and 11 Technical Subcommittees (TSs) addressed seismic hazard maps, structural design criteria and analysis, foundations, cast-in-place and precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, interface with codes and standards, and composite structures. Their work resulted in 58 substantive and 45 editorial proposals for change to the 1988 *Provisions*.

The PUC, under the leadership of Loring Wyllie of Degenkolb Enginners, approved more than 90 percent of the proposals and, in January 1991, the BSSC Board accepted the PUC-approved proposals for balloting by the BSSC member organizations in April-May 1991.
Following the balloting, the PUC considered the comments received with "yes with reservations" and "no" votes and prepared 21 reballot proposals for consideration by the BSSC member organizations. The reballoting was completed in August 1991 with the approval by the BSSC member organizations of 19 of the reballot proposals.

On the basis of the ballot and reballot results, the 1991 *Provisions* documents were prepared and transmitted to FEMA for publication in September 1991. Reports describing the changes made in the 1988 Edition and issues in need of attention in the next update cycle also were developed.

In August 1992, in response to a request from FEMA, the BSSC initiated an effort to continue its structured information dissemination and instruction/training effort aimed at stimulating widespread use of the *Provisions*. The primary objectives of the effort were to bring several of the publications complementing the *Provisions* into conformance with the 1991 Edition in a manner reflecting other related developments (e.g., the fact that all three model codes now include requirements based on the *Provisions*) and to bring instructional course materials currently being used in the BSSC seminar series (described below) into conformance with the 1991 *Provisions*.

The 1994 Edition

The effort to structure the 1994 PUC and its technical subcommittees was initiated in late 1991 chairing the OUC again for this cycle was Loring Wyllie. By early 1992, 12 Technical Subcommittees were established to address seismic hazard mapping, loads and analysis criteria, foundations and geotechnical considerations, cast-in-place and precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, interface with codes and standards, and composite steel and concrete structures, and base isolation/energy dissipation.

The TSs worked throughout 1992 and 1993 and, at a December 1994 meeting, the PUC voted to forward 52 proposals to the BSSC Board with its recommendation that they be submitted to the BSSC member organizations for balloting. Three proposals not approved by the PUC also were forwarded to the Board because 20 percent of the PUC members present at the meeting voted to do so. Subsequently, an additional proposal to address needed terminology changes also was developed and forwarded to the Board.

The Board subsequently accepted the PUC-approved proposals; it also accepted one of the proposals submitted under the "20 percent" rule but revised the proposal to be balloted as four separate items. The BSSC member organization balloting of the resulting 57 proposals occurred in March-May 1994, with 42 of the 54 voting member organizations submitting their ballots. Fifty-three of the proposals passed, and the ballot results and comments were reviewed by the PUC in July 1994. Twenty substantive changes that would require reballoting were identified. Of the four proposals that failed the ballot, three were withdrawn by the TS chairmen and one was substantially modified and also was accepted for reballoting. The BSSC Board of Direction accepted the PUC recommendations except in one case where it deemed comments to be persuasive and made an additional substantive change to be reballoted by the BSSC member organizations.

The second ballot package composed of 22 changes was considered by the BSSC member organizations in September-October 1994. The PUC then assessed the second ballot results and made its recommendations to the BSSC Board in November. One needed revision identified later was considered by the PUC Executive Committee in December. The final copy of the 1994 Edition of the *Provisions* including a summary of the differences between the 1991 and 1994 Editions was delivered to FEMA in March 1995.

The 1997 Edition

In September 1994, NIBS entered into a contract with FEMA for initiation of the 39-month BSSC 1997 *Provisions* update effort. Late in 1994, the BSSC member organization representatives and alternate representatives and the BSSC Board of Direction were asked to identify individuals to serve on the 1997 PUC and its TSs. The 1997 PUC, chaired by Bill Holmes of Rutherford and Chekene, was constituted early in 1995, and 12 PUC Technical Subcommittees were established to address design criteria and analysis, foundations and geotechnical considerations, cast-in-place/precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, interface with codes and standards, composite steel and concrete structures, energy dissipation and base isolation, and nonbuilding structures.

As part of this effort, the BSSC developed for the 1997 *Provisions* a revised seismic design procedure. Unlike the design procedure based on U.S. Geological Survey (USGS) peak acceleration and peak velocity-related acceleration ground motion maps developed in the 1970s and used in earlier editions of the *Provisions*, the new design procedure involves new design maps based on recently revised USGS spectral response maps and a process specified within the body of the *Provisions*. This task was conducted with the cooperation of the USGS (under a Memorandum of Understanding signed by the BSSC and USGS) by the Seismic Design Procedure Group (SDPG) working with the guidance of a five-member Management Committee.

More than 200 individuals participated in the 1997 update effort, and more than 165 substantive proposals for change were developed. A series of editorial/organizational changes also were made. All draft TS, SDPG, and PUC proposals for change were finalized in late February 1997, and in early March, the PUC Chair presented to the BSSC Board of Direction the PUC's recommendations concerning proposals for change to be submitted to the BSSC member organizations for balloting. The Board accepted these recommendations, and the first round of balloting was conducted in April-June 1997.

Of the 158 items on the first ballot, only 8 did not pass; however, many comments were submitted with "no" and "yes with reservations" votes. These comments were compiled for distribution to the PUC, which met in mid-July to review the comments, receive TS responses to the comments and recommendations for change, and formulate its recommendations concerning what items should be submitted to the BSSC member organizations for a second ballot. The PUC deliberations resulted in the decision to recommend to the BSSC Board that 28 items be included in the second ballot. The PUC Chair subsequently presented the PUC's recommendations to the Board, which accepted those recommendations.

The second round of balloting was completed in October. All but one proposal passed; however, a number of comments on virtually all the proposals were submitted with the ballots and were immediately compiled for consideration by the PUC. The PUC Executive Committee met in

December to formulate its recommendations to the Board, and the Board subsequently accepted those recommendations.

The PUC concluded its update work by identifying issues in need of consideration during the next update cycle and technical issues in need of study. The final version of the 1997 *Provisions*, including an appendix describing the differences between the 1994 and 1997 edition, was transmitted to FEMA in February 1998. The contract for the 1997 update effort was extended by FEMA to September 1999 to permit several complementary initiatives to be pursued.

One of these initiatives resulted in a CD that provides all of the design mapping data needed for use with the 1997 *NEHRP Recommended Provisions* and *International Building Code* as well as the *International Residential Code* and the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*. This CD was developed for the BSSC by Dr. E. V. Leyendecker of the U.S. Geological Survey. It permits the user to search either by longitude and latitude or by zipcode. Although the CD-ROM is distributed by FEMA and the BSSC, he International Code Council was given permission to reproduce copies to accompany the *International Building Code* (IBC) and *International Residential Code* (IRC).

The second initiative resulted in a list of the relevant seismic design map data on a county-by-county basis. One listing identifies populated places, state, county, population (when available), latitude and longitude, two maximum considered earthquake (MCE) spectral points (for use with the 1997 *NEHRP Recommended Provisions, International Building Code*: two spectral points for the 10 percent probability in 50 year maps (for use with the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*), and the corresponding category for use with the *International Residential Code*. The final version of this listing can be sorted alphabetically by county and then by place in the county. Another listing presents the counties for each state and provides the same information as in the first listing but uses the approximate geographic or "centroid" coordinates to determine the data grid values for each county as a whole. These listings are based on the USGA developed CD and were assembled for the BSSC by Richard McConnell.

In a somewhat related effort, the BSSC commissioned a set of approximately 40 comparative designs. Each comparative design was performed at least three times: once according to the proposed 2000 *IBC* (which is being take to represent the 1997 *NEHRP Recommended Provisions*), once according to the 1991 *Provisions* (requirements reflected in the *National Building Code* and *Standard Building Code*), and once according to the 1994 *Uniform Building Code*. Performing the study for the BSSC were the J. R. Harris and Company and S. K. Ghosh Associates, Inc.

Also developed during this update cycle was the BSSC website ----www.bssconline.org . The site provides BSSC with a means for posting proposals for changes and other information for public comment and also is a venue for a host of downloadable material including the *Provisions* and *Commentary.*

The 2000 Edition

In September 1997, NIBS entered into a contract with FEMA for initiation of the 48-month BSSC effort to update the 1997 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*.

In lieu of the Seismic Design Procedure Group (SDPG) used in the 1997 update, the BSSC re-established Technical Subcommittee 1, Seismic Design Mapping, used in earlier updates of the *Provisions*. This subcommittee is composed of an equal number of representatives from the earth science community, including representatives from the USGS, and the engineering community.

An additional 11 subcommittees were formed to address seismic design and analysis, foundations and geotechnical considerations, cast-in-place and precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, composite steel and concrete structures, base isolation and energy dissipation, and nonbuilding structures. Two ad hoc task groups also were formed: one to develop appropriate anchorage requirements for concrete/masonry/wood elements and the other to develop a simplified procedure for use in the lower seismic risk areas. No technical subcommittee was established in this update cycle to serve specifically as the interface with codes and standards; rather, the BSSC's Code Resource Support Committee provided for the needed liaison between the PUC and the model code and standards organizations.

The first ballot encompassing 146 proposals for change to the 1997 *Provisions was* submitted to the BSSC member organizations in April, 2000; the ballot deadline in June. The proposals for change also were posted for comment on the BSSC website. Of the 64 member organizations who received ballot packages, 42 responded. Of the 146 proposals, 69 passed with no "no" votes but some "yes with reservations" votes, 71 passed but with "no" and "yes with reservations" votes, and 6 did not pass (i.e., received less than 67 percent "yes" and "yes with reservations" votes). The comments submitted with "no" and "yes with reservations" votes were compiled and distributed to the PUC Technical Subcommittee chairs. The PUC then met in Denver in July 2000 to receive the TSs responses to ballot comments and formulate recommendations concerning items that need to be submitted to the member organizations for a second ballot

In August 2000, PUC Chair William Holmes briefed the BSSC Board of Direction on the results of the first ballot and recommended that 17 items be submitted to the membership for a second ballot. Ten of the proposals were revisions of previous proposals, three were new proposals, and four were proposals developed by the PUC to clarify concerns arising from the first ballot.

The official second ballot package was mailed to BSSC member organizations for voting in September- October 2000. Of the 66 BSSC member organizations, 42 responded and all proposals passed. There were, however, several "yes with reservations" and "no" votes, and the PUC met on October 30-31, 2000, to resolve the comments submitted with these votes and to formulate recommendations concerning a third ballot.

On November 1, 2000, the PUC chair presented the second ballot results to the BSSC Board and recommended that several items be submitted to the membership for a third ballot. The primary purpose of the third ballot was to permit integration into the 2000 *Provisions* of new steel requirements resulting from the FEMA-funded SAC effort mounted to study damage during the Northridge earthquake and of the most current version of the American Institute of Steel Constructions standard which was expected to include many of the SAC requirements. The third ballot, which included five proposals, was sent to the membership for vote by February 2001. Of the 65 member organizations, 44 submitted ballots (67 percent). All five proposals passed and the results were reviewed and comments resolved by the PUC Executive Committee at a meeting in March 2001.

The PUC chair briefed the BSSC Board on the third ballot results on March 6, 2000, and the Board unanimously approved the 2000 *Provisions* for transmittal to FEMA following a final editorial review by the PUC of the *Provisions* document and its accompanying *Commentary* volume. Reports identifying the major differences between the 1997 and the 2000 Editions of the *Provisions* and describing unresolved issues and major technical topics in need of further study also were prepared. Code-language versions of changes for the 2000 *Provisions* for submittal as proposed code changes for the 2003 Edition of the *IBC* were developed for the BSSC by S. K. Ghosh Associates.

The 2003 Edition

Well before the actual contract between FEMA and the BSSC was awarded, planning for the 2003 Edition was under way. Several major where the initial topics of attention. First, in January 2001a meeting was held to decide how best to handle the diverse subject of nonbuilding structures. It was concluded that the best solution for the 2003 cycle was to recommend that the nonbuilding structures technical subcommittee (T S 13) continue but have greater representation on the PUC with four members. It was also recommended that TS 13 form eight subgroups to address major nonbuilding structures categories such as chimneys, wharves and piers, tanks and vessels, etc.

The second area of concern was a detailed edit of the 2000 provisions to eliminate the undue repetition and inconsistencies that had crept in over the years. This edit performed at the end of the 2000 cycle by Michael valley of Magnusson and Klemencic. After a thorough review of the edited document, this "Reformatted" version was voted on by the BSSC membership in October 2001. It was accepted and became the basis for the 2003 update.

The final issue involved structuring the 2003 update to reflect the fact that the *Provisions* requirements were being reflected in ASCE 7, the IBC, the IRC, and the NFPA 5000. Further it seemed likely that the I codes would cover most seismic matters by referencing the ASCE 7. Thus it appeared most reasonable to coordinate the BSSC efforts with those of ASCE 7 Seismic Task Committee, there by relieving the PUC and its TSs of the responsibility for maintaining code language. Considerable progress has been made on integrating ASCE 7 as a full reference standard during the 2003 update cycle and it is expected that this effort will be completed during the next update.

The proposal for the 2003 update of the NEHRP Recommended Provisions was submitted to FEMA in June 2001. In order to keep the momentum of the update process and with the concurrence of the FEMA Project Officer, candidates for update committee membership were identified and recommended membership lists were reviewed and accepted by the BSSC Board at a June 2001 meeting along with a revised procedures/goals statement for the effort developed to reflect the thoughts expressed at the BSSC Annual Meeting in March. Letters of invitation to serve on the update committees were mailed in late June 2001.

The 2003 Provisions Update Committee (PUC) convened for the first time in July in conjunction with a meeting of the Joint Correlating Committee, which was established for the 2003 update cycle to eliminate duplication of efforts by those working on the Provisions and those working on ASCE 7. The PUC Technical Subcommittee (TS) chairs identified topics they intended to consider during the update and a tentative schedule for the project was established.

FEMA signed the contract with NIBS for the 2003 update project on September 28, 2001. This 30-month contract provides for conduct of a base series of tasks and two options.

A comprehensive edit of the 2000 *Provisions* initiated in early 2000 to eliminate undue repetition and inconsistencies and generally make the document more user-friendly was completed in late summer and reviewed by the PUC. After revisions to reflect PUC member comments, the draft reformatted document was accepted by the BSSC Board for balloting by the BSSC member organizations to determine whether the revised draft could be used as the base document for the remainder of the 2003 update effort. The balloting occurred between October and December 2001. Forty of the 65 BSSC member organizations submitted ballots on the reformatted Provisions and the document was approved; however, a number of significant comments accompanied the ballots. These comments were compiled and responses formulated. In January 2002, the PUC debated resolution of the comments and submitted its recommendations to the BSSC Board, which accepted the PUC recommendations and approved use of the reformatted 2000 Provisions as the base document for the remainder of the 2003 update effort.

Work also began in early 2002 on development of the new BSSC website that is expected to permit Technical Subcommittee and PUC members to develop, review, and vote on proposals in an interactive electronic environment and that also will permit the BSSC member organizations to receive proposals and submit their ballots electronically.

Proposals for change to be submitted to the PUC for ballot were submitted in late August 2002 and were mailed to the PUC for balloting on September 11, 2002. Completed ballots were due in mid-October, and the results were compiled for review/response by the relevant Technical Subcommittee in preparation for review by the full PUC. The PUC then met in Washington, D.C., on November 7-8 and formulated its recommendations for the BSSC Board concerning proposals to be submitted to the BSSC member organizations for ballot. Of the 77 proposals initially submitted by its technical subcommittees, the PUC recommended to the Board that 54 proposals be submitted to the BSSC member organizations for ballot but that this balloting not occur until all proposals for change for the 2003 Provisions are completed. The Board accepted this recommendation, and remaining proposals were scheduled to be submitted for PUC review by April 1, 2003.

Approximately 90 proposals were submitted for a mail ballot by the PUC. This balloting was completed in early June and the PUC met on June 15-17, 2003, to resolve comments and formulate its recommendations concerning which of this second batch of proposals should be submitted to the membership. The BSSC Board received and accepted the PUC recommendations on June 18.

Ninety-nine new proposals (those submitted to the PUC by mail plus a number of PUC proposals developed at the meeting) were reviewed and voted on by the PUC at a three-day meeting held in San Diego, California, in June15-17, 2003. Of these, 84 were accepted by the PUC, many with revisions, and subsequently submitted to the BSSC Board with the recommendation that they be added to the 54 proposals approved earlier and submitted to the BSSC member organizations for ballot. The Board accepted the PUC recommendation and the ballot package (composed of the ballot sheet, proposals, composites of the reformatted Provisions and Commentary, and the comments and responses on each proposal) was sent to the representatives and alternates of the 63 BSSC member organizations on August 1, 2003. Ballots were due October 1.

The member organization votes were tallied and comments were forwarded to the appropriate PUC technical subcommittee chairs in mid-October in preparation for a November meeting of the PUC at which ballot comments were addressed. Given that the contract with FEMA requires delivery of the consensus approved 2003 *Provisions* and *Commentary* in March 2004, another ballot will not be possible; therefore, the BSSC Board authorized the PUC to resolve, if possible, comments on proposals that have passed the membership ballot and to consider any proposals for which comments cannot be resolved as items for reconsideration in the next update cycle.

The PUC met on November 20-21, 2003, to review the proposals for change. Approximately 130 proposals received the required two-thirds affirmative votes; with approximately half of those requiring some revisions in response to comments. On November 22, the PUC chair presented the results to the BSSC Board. The Board addressed two contentious issues at the request of the PUC and accepted the PUC recommendations regarding the changes to be made for the 2003 edition of the *Provisions* and *Commentary*. The final draft is now being assembled for review by the PUC and it will be officially delivered to FEMA by the end of April 2004.

Planning for the next update cycle is beginning and a small task group met on November 19, 2003, to discuss how to structure the next *Provisions* update cycle to adopt ASCE 7 by reference. It also appears that the PUC will be somewhat smaller in the next cycle and there will be fewer technical subcommittees. Ad hoc issue committees will be appointed on an as-needed basis to address research needs and develop emerging technologies. Coupled with this streamlining, it is anticipated that the next edition of the *Provisions* will be issued in 2008, rather than 2006, to better mesh with the codes and standards development schedules.

CODE RESOURCE DEVELOPMENT AND SUPPORT

In mid-1996, FEMA asked the BSSC to initiate an effort to generate a code resource document based on the 1997 *Provisions* for use by the International Code Council (ICC) in adopting seismic provisions for the first edition of the *International Building Code (IBC)* to be published in 2000. The Code Resource Development Committee (CRDC) appointed to conduct this effort met several times over the next year and the CRDC-developed draft requirements were presented to the ICC's *IBC* Structural Subcommittee in March 1997.

Subsequently, the CRDC met to develop comments on the *IBC* working draft to be submitted to the ICC in preparation for an August 1997 public comment forum. These comments generally reflected actions taken by the PUC in response to comments submitted with the first ballot on the changes proposed for the 1997 *Provisions* as well as CRDC recommendations concerning changes made by the *IBC* Structural Subcommittee in the original CRDC submittal. CRDC representatives attended the August forum to support the CRDC recommendations.

After issuance of the first draft of the *IBC* in November 1997, the CRDC met to prepare "code change proposals" that reflected the final version of the 1997 *Provisions* for submittal in January 1998. The CRDC then met for the last time as a committee in March 1998 to review the compilation of *IBC* code change proposals issued by the ICC and to develop a strategy for supporting the code change proposals it had developed at an *IBC* public hearing in April. In addition, the *IBC* Structural Subcommittee asked for CRDC input concerning all the seismic-related code change proposals and these comments were summarized and transmitted to the *IBC* group for its consideration.

An eight-member Code Resource Support Committee (CRSC) then was established to support the *Provisions*-based requirements through the remainder of the adoption process and to provide for needed liaison with the 2000 *Provisions* development work. A CRSC Technical Advisory Group (TAG) composed of representatives of the 2000 PUC and the various materials interests also was established to support the CRSC. The first task of the CRSC was to deal with one major issue that arose at the April hearing at which several code change proposals concerning the draft *IBC* (and 1997 *Provisions* based) response modification factors and limits of applicability of certain structural systems were discussed. At the suggestion of a CRDC representative at the hearing, the proponents of those code changes agreed to withdraw their proposals to permit discussion of their technical merit outside the forum of the public hearing process. To this end, the CRSC invited these code change proponents as well as representatives of the various construction industry materials associations to an August 1998 meeting at which the group formulated a consensus opinion on an appropriate series of code change proposals that could be submitted to replace those withdrawn in April. Additional topics also were discussed and a total of 13 code-change proposals were drafted.

In September 1998, the 2000 PUC Executive Committee was briefed on these code-change proposals, most of which were accepted by the PUC as items to be considered during the 2000 update effort; however, five items were deemed to be significant departures from the 1997 *Provisions* and required a vote by the full PUC. This balloting concluded in early October with all items achieving consensus approval. The CRSC then finalized all 13 of its code change proposals and submitted them to the ICC in late October 1998.

In January and February 1999, the CRSC met with its Technical Group to consider the proposed changes to the *International Building Code* seismic provisions that would be debated at March 1999 hearings. The CRSC chair and several member participated in the hearings on behalf of the CRSC.

An *International Residential Code* Task Group established within the CRDC in late-1997 has provided the ICC committee developing the *International Residential Dwelling Code* (*IRC*) with input concerning seismic requirements reflecting the 1997 *Provisions*, and these requirements generally were reflected in the draft *IRC*. The activities of this task group have paralleled those of the CRDC/CRSC with the *IBC* and representatives attended the *IRC* July 1998 public hearing in Kansas City. At this hearing, agreement was reached on the seismic map to be included in the *IRC*; this map subsequently was prepared for the BSSC by USGS and submitted to the ICC for inclusion in the final draft of the *IRC*. The task group met in February 1999 to review proposed code changes and prepare for the March ICC hearings.

The CRSC chair and several CRSC members represented the group at the joint annual conference of BOCA, ICBO, and SBCCI held in September 1999 in St. Louis. Overall, the CRSC was successful in that almost all challenges to the seismic provisions were decided in favor of the CRSC position and the seismic provisions in both the 2000 *International Building Code* and the *International Residential Code* reflect the 1997 *NEHRP Recommended Provisions*.

In preparation for the ICC hearings to be held in Birmingham, Alabama, in April 2000, the CRSC and its Technical Group reviewed the code changes and met via telephone conference calls in March 2000 to discuss the proposals. The CRSC chair and several other CRSC members attended the hearings. With respect to the *International Building Code*, the CRSC had specific positions on 41 proposals. Of these proposals, 35 were decided in the direction CRSC favored and two that the CRSC opposed were withdrawn. During the hearings on the *International Residential Code*, the CRSC had specific positions on 12 proposals. Eight of these proposals were decided in favor of CRSC's position and one was withdrawn at CRSC's request.

In late September 2000, NIBS entered into a contract with FEMA to fund further code support work by the BSSC. Thus, the 2001 CRSC was reconstituted to include additional members and two special task groups; one to focus on the *IRC*, and one to focus on the NFPA code. The expanded CRSC and its Technical Advisory Group (TAG) reviewed the proposals for change to the *IBC* and *IRC* in preparation for the hearing held in Portland, Oregon, in late March 2001. During a February 23 conference call, the CRSC formulated its position on the proposed changes to the *IRC*. At a meeting on March 5, with its TAG, the CRSC decided upon its positions on the proposed changes to the *IBC*. The CRSC chair and several CRSC members attended the hearing.

The CRSC's NFPA Task Group members attended meetings of the NFPA Technical Correlating Committee (TCC) and Structures and Construction Committee. In addition, the CRSC representative to the TCC has been appointed by that committee as its representative to the Performance Task Group to the Fundamentals Committee.

The 2001 CRSC met in Denver in July 2001 to review draft code change proposals based on the changes made for the 2000 *Provisions*. As noted above, S. K. Ghosh Associates, Inc., prepared drafts of the *IBC*-related proposals and Kelly Cobeen and Alan Robinson developed the *IRC* proposals. These proposals were then revised in response to CRSC comments and, as directed by the BSSC Board, sent to the BSSC member organizations for comment.

The CRSC met in October 2001 to review the comments received and to address other code-related matters including the need for additional changes identified during work on ASCE 7 and CRSC work on NFPA 5000. CRSC-approved code changes then were officially submitted to the ICC in November.

Several CRSC members presented an educational workshop on the enforcement implications for code officials of adoption of the *IBC/IRC* at the BOCA Annual Business Meeting in September 2001.

The CRSC initiated its review of those proposals for change to the *IBC* and *IRC* affecting seismic matters in mid-February 2002 in preparation for a mid-March meeting at which the group formulated its official position on these proposals and identified which CRSC members would represent the committee at the ICC hearings scheduled for April 2002. Several members of the CRSC attended the hearings and, overall, the group's positions on specific changes tended to prevail.

In mid-September 2002, the CRSC convened via telephone to review the final action agenda for the ICC Codes Forum to be held in Ft. Worth, Texas, the first week in October. Plans were also made for CRSC representation at the meeting.

In early March, BSSC was informed that a proposed educational workshop on the enforcement implications for code officials of adoption of the *IBC/IRC* had been accepted for presentation at the Ft. Worth meeting and in mid-September the individuals involved in the presentation convened via telephone to refine plans for the presentation.

Several CRSC members attended the ICC Codes Forum in Fort Worth in October 2002. The educational session on *IBC/IRC* code enforcement implications also was conducted twice and was well received. During late 2002, CRSC representatives attended code adoption meetings in Kentucky and South Carolina.

The CRSC met in February 2003 to review already-accepted proposals for change for the 2003 *Provisions* to determine whether any should be submitted as proposals for change for the *International Building Code* or *International Residential Code*. The group decided that it would submit only one proposal for change – i.e., one that would change the map for the *International Residential Code* in the central states and the southeast to reduce the area in which substantial seismic requirements would prevail.

The CRSC also has nominated several individuals to represent CRSC/FEMA interests on several technical committees involved in the National Fire Protection Association *NFPA 5000* code change process. At FEMA's request, the CRSC also nominated an individual to represent CRSC/FEMA interests on the NFPA committee responsible for two manufactured housing standards.

The CRSC met in June 2003 in conjunction with the BSSC Annual Meeting and formulated plans for review of proposals in preparation for the ICC hearings in September and for an effort to develop a change proposal for submittal to the ASCE 7 Seismic Task Committee that will present a reformatted version of the ASCE 7 seismic requirements intended to be more user friendly and to reflect the reformatting work done for the 2003 edition of the *NEHRP Recommended Provisions*.

Following individual review and input concerning the proposals for change to the *IBC* and *IRC* affecting the seismic requirements, the CRSC convened via telephone in August to determine the positions to be taken by the CRSC representatives who would attend the hearings. Subsequently, six CRSC members represented the group at the *IBC* portion of the hearings and three, at the *IRC* portion. As has consistently been the case, the positions taken by the CRSC tend to prevail with the relevant ICC committees overseeing the hearings.

In July 2003, a representative of the CRSC participated in a hearing on *IBC* adoption in the state of Tennessee. In addition, cost information developed earlier to show the impact of the 2000 *IBC* seismic requirements on costs of typical buildings over the costs for the same structures constructed under prevailing codes in a number of geographic areas was provided to individuals in Tennessee. Planning is under way for the development of additional designs that will compare the cost impact of the *IBC* seismic requirements over the prevailing code requirements for typical buildings in the central and southern states.

Development of the code change proposal for ASCE 7 was completed in January and officially submitted to the ASCE 7 Seismic Task Committee. In addition, the CRSC convened in March 2004 to review the public comments on ICC code change proposals. Few of the comments focused on seismic issues; consequently, only two CRSC representatives will attend the ICC meeting in May. CRSC representatives continue to serve on a number of NFPA 5000 technical committees and CRSC participants have been helpful in drafting a proposal on anchorage of manufactured housing to resist earthquake ground motions..

FEMA also has entered into a new contract with NIBS to support the BSSC's codes and standards work through FY 2004 and, through options, through FY 2006. FEMA currently is considering the BSSC proposal for funding for the BSSC's code development and support functions through FY 2004.

INFORMATION DISSEMINATION

The BSSC continues in its efforts to stimulate widespread use of the *Provisions*. In addition to the issuance of a variety of publications that complement the *Provisions*, over the past decade the BSSC has developed materials for use in and promoted the conduct of a series of seminars on application of the *Provisions* among relevant professional associations.

In September 1997, NIBS entered into a 60-month indefinite quantity contract with FEMA for conduct of the BSSC's information dissemination. The first task orders issued under the contract charge the BSSC to increase its capability to respond to requests for technical assistance relating to the *Provisions*, to increase its capability to provide more general technical assistance and information in a coordinated and proactive manner and using all communication media including its website, to review existing complementary publications and educational seminar materials not already revised in whole or in part to reflect the 1997 *Provisions* and to prepare a plan to bring them into conformance with the substantive content of the 1997 *Provisions* if such is deemed appropriate or to develop different documents aimed at changing audiences, to revise the course materials including the *Guide to Application of the Provisions*, an instructors manual and slide set, and a student manual to reflect the 1997 *Provisions* and the code requirements based on the *Provisions*, to prepare and implement a plan to market the instructional materials and subsequently conduct an ongoing series of instructional (both technical and nontechnical) training seminars on an as-requested basis, to continue to promote and encourage the use of the *Provisions* by the nation's model code organizations and their adoption by local jurisdictions, and to continue to conduct activities to increase the general awareness of the earthquake risks in different regions throughout the country and the need to use local building codes that are substantially equivalent with the *Provisions*.

Because of unanticipated delays in preparation of the new *Guide* and instructional materials, the decision was made in early 2001 that the work should focus instead on the 2000 edition of the *Provisions*. Since that time, the *Guide* document has been developed, finalized, and reviewed by the BSSC subcontractor conducting the project and the instructional materials have been pilot tested in several venues including the 2001 and 2002 Multihazard Building Summer Design Institutes held in July 2001 and 2002 at the Emergency Management Institute.

The final draft of the new *Guide to Application of the 2000 NEHRP Recommended Provisions* was received in autumn 2002 and was sent to the Provisions Update Committee, the group possessing the greatest in-depth understanding of the *Provisions*, and selected BSSC Board members for review. This review was completed in March 2003 and changes were made in response to reviewer comments. The final draft was then submitted by the BSSC subcontractor in April and the material was pilot tested in various venues (including the Emergency Management Institute's Multihazard Building Design Summer Institute in July-August 2003).

The final copy edit of the *Guide* was completed in October 2003 and an effort was mounted to "extend the life" of the document, originally prepared to reflect the 2000 edition of the *Provisions*, by integrating cross references to the relevant section numbers in the 2003 *Provisions* and by integrating notes that focus on changes that will be made for the 2003 *Provisions* (mention of those proposals that are not approved will be removed before FEMA printing of the document). With the final decisions now made about the 2003 *Provisions*, this effort will be completed by the end of April 2004.

Funded by FEMA in September 2002 was an 18-month effort to develop an up-to-date version of an earlier FEMA publication, *Home Builders' Guide to Seismic-Resistant Construction*, and to update an earlier BSSC publication, *Nontechnical Explanation of the NEHRP Recommended Provisions*, to reflect the 2000/2003 *Provisions*. Work on both these documents is well under way and a briefing on the plans for the *Home Builders'* was presented at the BSSC Annual Meeting in June. These two documents in combination with the *Guide to Application* and associated educational materials provide the resources needed to familiarize a large segment of the building community with the *Provisions*.

In September 2003, FEMA issued an additional task order to fund BSSC information dissemination efforts through FY 2004.

IMPROVING THE SEISMIC SAFETY OF EXISTING BUILDINGS

Guidelines/Commentary Development Project

The 1997 *NEHRP Guidelines for the Seismic Rehabilitation* and *Commentary* volumes and 1997 map packet (which also include maps referenced in the *NEHRP Recommended Provisions for New Buildings and Other Structures*) are readily available as are two companion volumes – *Planning for Seismic Rehabilitation: Societal Issues* (FEMA 275) and *Example Applications of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 276).

Case Studies Project

The case studies project was an extension of the multi-year project leading to publication of the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* and its *Commentary* in late 1997. The project is expected to contribute to the credibility of the *Guidelines* by providing potential users with representative real-world application data and to provide FEMA with the information needed to determine whether and when to update the *Guidelines*. The final report on the project was delivered to FEMA in September 1999 and is now available as FEMA 343, *Case Studies: An Assessment of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings.*

Guidelines Training Seminars

In August 1997, NIBS entered into a contract with FEMA for the design and conduct of a series of technical training seminars to transfer the technology and information contained in the *Guidelines* to structural and architectural engineers (whether in private or government practice, representing organizations both large and small); to local building officials and technical staffs, interested contractors, and mitigation officials, where applicable; and to engineering educators and students in institutions offering seismic design curricula. Conceptually, the seminar curriculum will take the form of a series of modules that will permit it to be adapted for use with a variety of audiences.

The Applied Technology Council, under contract to the BSSC, developed the seminar program syllabus and other instructional materials. To date, approximately 2000 structural engineers have attended seminars on the *NEHRP Guidelines for \the Seismic Rehabilitation of Buildings*. Being conducted for FEMA by the BSSC with the assistance of the Applied Technology Council, two-day seminars have been held in San Diego; Salt Lake City; Portland, Oregon; Los Angeles; Seattle; New York City; Oakland; St. Louis; Charleston, South Carolina; Chicago, Illinois; Sacramento, California; and Washington, D.C.

BSSC MEMBER ORGANIZATIONS
(* indicates affiliate nonvoting member)

AFL-CIO Building and Construction Trades Department.
American Concrete Institute
American Consulting Engineers Council
American Forest and Paper Association
American Institute of Architects
American Institute of Steel Construction
American Iron and Steel Institute
American Society of Civil Engineers
American Society of Civil Engineers--Kansas City Chapter
American Society of Mechanical Engineers
American Welding Society
APA - The Engineered Wood Association
Applied Technology Council
ASHRAE ,Inc.
Associated General Contractors of America
Association of Engineering Geologists
Association of Major City Building Officials
Bay Area Structural, Inc.*
Brick Industry Association
Building Owners and Managers Association International
Building Technology, Incorporated*
California Geotechnical Engineers Association
California Seismic Safety Commission
Canadian National Committee on Earthquake Engineering
City of Hayward, California*
Concrete Masonry Association of California and Nevada
Concrete Reinforcing Steel Institute
Concrete Reinforcing Steel Institute
Division of state Architect (California)
Earthquake Engineering Research Institute
Felten Engineering Group, Inc.*
General Services Administration Seismic Program
Hawaii State Earthquake Advisory Board
H&H Group, Inc.*
HLM Design*
Institute for Business and Home Safety
Interagency Committee on Seismic Safety in Construction
International Code Council
International Masonry Institute
LaPay Consulting, Inc.*
Masonry Institute of America
Metal Building Manufacturers Association
Mid-America Earthquake Center
National Association of Home Builders
National Concrete Masonry Association
National Conference of States on Building Codes and Standards
National Council of Structural Engineers Associations
National Elevator Industry, Inc.
National Fire Sprinkler Association
National Institute of Building Sciences
National Ready Mixed Concrete Association
Portland Cement Association
Precast/Prestressed Concrete Institute
Rack Manufacturers Institute
Santa Clara University
Square D Company*
Steel Deck Institute, Inc.
Steel Joist Institute*
Structural Engineers Association of California
Structural Engineers Association of Central California
Structural Engineers Association of Colorado
Structural Engineers Association of Illinois
Structural Engineers Association of Kentucky
Structural Engineers Association of Northern California
Structural Engineers Association of Oregon
Structural Engineers Association of San Diego
Structural Engineers Association of Southern California
Structural Engineers Association of Texas
Structural Engineers Association of Utah
Structural Engineers Association of Washington
The Masonry Society
U.S. Army CERL
Vibration Mountings and Controls*
Western States Clay Products Association
Western States Structural Engineers Association
Wire Reinforcement Institute, Inc.

BUILDING SEISMIC SAFETY COUNCIL
PUBLICATIONS

Available free from the Federal Emergency Management Agency at 1-800-480-2520 (order by FEMA Publication Number). For detailed information about the BSSC and its projects, contact: BSSC, 1090 Vermont Avenue, N.W., Suite 700, Washington, D.C. 20005 Phone 202-289-7800; Fax 202-289-1092; e-mail ctanner@nibs.org

NEW BUILDINGS PUBLICATIONS

The NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings, 2003 Edition, 2 volumes and maps, FEMA 450 (issued as a CD with only limited paper copies available).

The NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings, 2000 Edition, 2 volumes and maps, FEMA 368 and 369

Guide to Application of the 1991 Edition of the NEHRP Recommended Provisions in Earthquake Resistant Building Design, Revised Edition, 1995, – new edition to be issued as FEMA 451 in preparation

A Nontechnical Explanation of the NEHRP Recommended Provisions, Revised Edition, 1995, FEMA 99 – new edition in preparation.

Seismic Considerations for Communities at Risk, Revised Edition, 1995, FEMA 83 – new edition expected to be published in late 1999 or early 2000

Seismic Considerations: Apartment Buildings, Revised Edition, 1996, FEMA 152

Seismic Considerations: Elementary and Secondary Schools, Revised Edition, 1990, FEMA 149

Seismic Considerations: Health Care Facilities, Revised Edition, 1990, FEMA 150

Seismic Considerations: Hotels and Motels, Revised Edition, 1990, FEMA 151

Seismic Considerations: Office Buildings, Revised Edition, 1996, FEMA 153

Societal Implications: Selected Readings, 1985, FEMA 84

EXISTING BUILDINGS

NEHRP Guidelines for the Seismic Rehabilitation of Buildings, 1997, FEMA 273

NEHRP Guidelines for the Seismic Rehabilitation of Buildings: Commentary, 1997, FEMA 274

Case Studies: An Assessment of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings, 1999, FEMA 343

Planning for Seismic Rehabilitation: Societal Issues, 1998, FEMA 275

Example Applications of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings, 1999, FEMA 276

NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings, 1992, FEMA 172

NEHRP Handbook for the Seismic Evaluation of Existing Buildings, 1992, FEMA 178

An Action Plan for Reducing Earthquake Hazards of Existing Buildings, 1985, FEMA 90

MULTIHAZARD

An Integrated Approach to Natural Hazard Risk Mitigation, 1995, FEMA 261/2-95

LIFELINES

Abatement of Seismic Hazards to Lifelines: An Action Plan, 1987, FEMA 142

Abatement of Seismic Hazards to Lifelines: Proceedings of a Workshop on Development of An Action Plan, 6 volumes:

Papers on Water and Sewer Lifelines, 1987, FEMA 135

Papers on Transportation Lifelines, 1987, FEMA 136

Papers on Communication Lifelines, 1987, FEMA 137

Papers on Power Lifelines, 1987, FEMA 138

Papers on Gas and Liquid Fuel Lifelines, 1987, FEMA 139

Papers on Political, Economic, Social, Legal, and Regulatory Issues and General Workshop Presentations, 1987, FEMA 143

(February 2004)