U.S. Seismic Steel Codes

EGOR P. POPOV

Major changes and recommendations for the seismic design of steel buildings were recently introduced into several U.S. design codes and specifications. Many of these changes deal primarily with material and detailing requirements. However, these problems cannot be separated from the underlying issues of lateral seismic loads. Therefore in this paper a broader point of view is adopted, and a brief discussion of relevant findings from seismology and geotechnical engineering is presented first. A general discussion of structural code developments and the relationships among the lateral load requirements in different codes follows. Specific issues pertaining to the seismic design of structures are then brought in. Limitations of the currently dominant elastic methods of analysis and design for seismic resistant structures are critically examined. The paper concludes with suggestions for future research.

SEISMOLOGICAL SETTING

Earthquakes occur throughout the world. The small dots on the world seismicity map shown in Fig. 1 indicate the epicenters, i.e., points on the earth's surface vertically above the focus of origins of recorded major earthquakes. Note in particular the concentration of such points along the circum-pacific land masses, and the large amount of activity in the Near East, and in Central Asia. Not to be overlooked are the dots representing recorded earthquakes in the middle of the oceans. It is believed that an assembly of the continents formed a single cluster called Pangaea millions of years ago. They have been drifting apart since. "The Restless Earth," aptly so called by Calder (1972), is constantly in motion, and the main cause of earthquakes is the movement of huge tectonic plates into which the thin earth crust is subdivided.

The typical mechanism causing earthquakes is illustrated in Fig. 2 for the 1989 Loma Prieta earthquake (Plafker and Galloway, 1989). As one tectonic plate, such as the Pacific plate in the figure, very slowly moves past another, such as the North American plate, tremendous shear strain is accumulated at their juncture. On reaching critical stress, the earth crust ruptures. In the case cited the focal point of the rupture occurred at a hypocenter 11.5 miles (18.4 km) below the epicenter. In the immediate proximity of the hypocenter the relative movement between the tectonic plates was 6.2 ft (1.9 m) in the horizontal direction, and 4.3 ft (1.3 m) in the vertical direction. Examples of such earth movements are many. One is shown in Fig. 3. Numerous similar cases of large movements of the earth's crust due to earthquakes can be cited (San Francisco 1906, Kanto 1923, Philippines 1990, Armenia 1988, Mexico City 1985, Alaska 1964, etc.). Great damage can occur to man-made structures on and in the proximity of such displacements.

Major earthquakes resulting from slip along a fault plane are commonly measured by a Richter magnitude. This is determined by measuring the amplitude of the ground motion of the seismic waves at several seismographic stations. The magnitude scale being logarithmic, an earthquake of magnitude 8 has 10 times the amplitude of a quake of magnitude 7, and 100 times the amplitude of an event of magnitude 6. Moreover an earthquake of magnitude 8 radiates over 30 times the energy of a quake of magnitude 7, and approximately 1000 times the energy of an earthquake of magnitude 6. Stronger earthquakes rupture a fault for a few hundred miles and last longer, the opposite is true for smaller earthquakes. The great 1906 San Francisco earthquake of Richter magnitude $M = 8.3$ had a fault rupture length of 400 km (250 miles), see Fig. 4; the Loma Prieta earthquake of magnitude $M = 7.1$ had a rupture of only 40 km (25 miles).

Whereas globally it is convenient to refer to earthquakes by their Richter magnitude, from an engineering point of view contour lines showing local intensities are very useful. Two such plots are shown in Figs. 5 and 6. The isoseismal lines for the great 1906 San Francisco earthquake are shown in Fig. 5 using the Rossi/Forel intensity scale ranging from a minimum of I to a maximum of XII. In the U.S. this scale is now largely replaced by the modified Mercalli Intensity (MMI) scale ranging from a low of I to a maximum of X. The diagram in Fig. 6 for the 1989 Loma Prieta earthquake is in the MMI scale. Other seismic intensity scales are used in other countries. All of these intensity scales are subjective, but experience shows them to be very meaningful. All of them assign the smallest number, such as I, to a barely perceptible shake, and the largest number, such as X or XII, to total damage.

Seismology provides engineers with very valuable information in the form of accelerograms. These depend on the locations where measurements are taken. Some representa-

This paper, with minor modifications, is reprinted with permission from *Proceedings of the Fourth World Congress on Tall Buildings and Urban Habitat*, Hong Kong, November 1990.

Egor P. Popov is professor emeritus of civil engineering at University of California, Berkeley, CA.

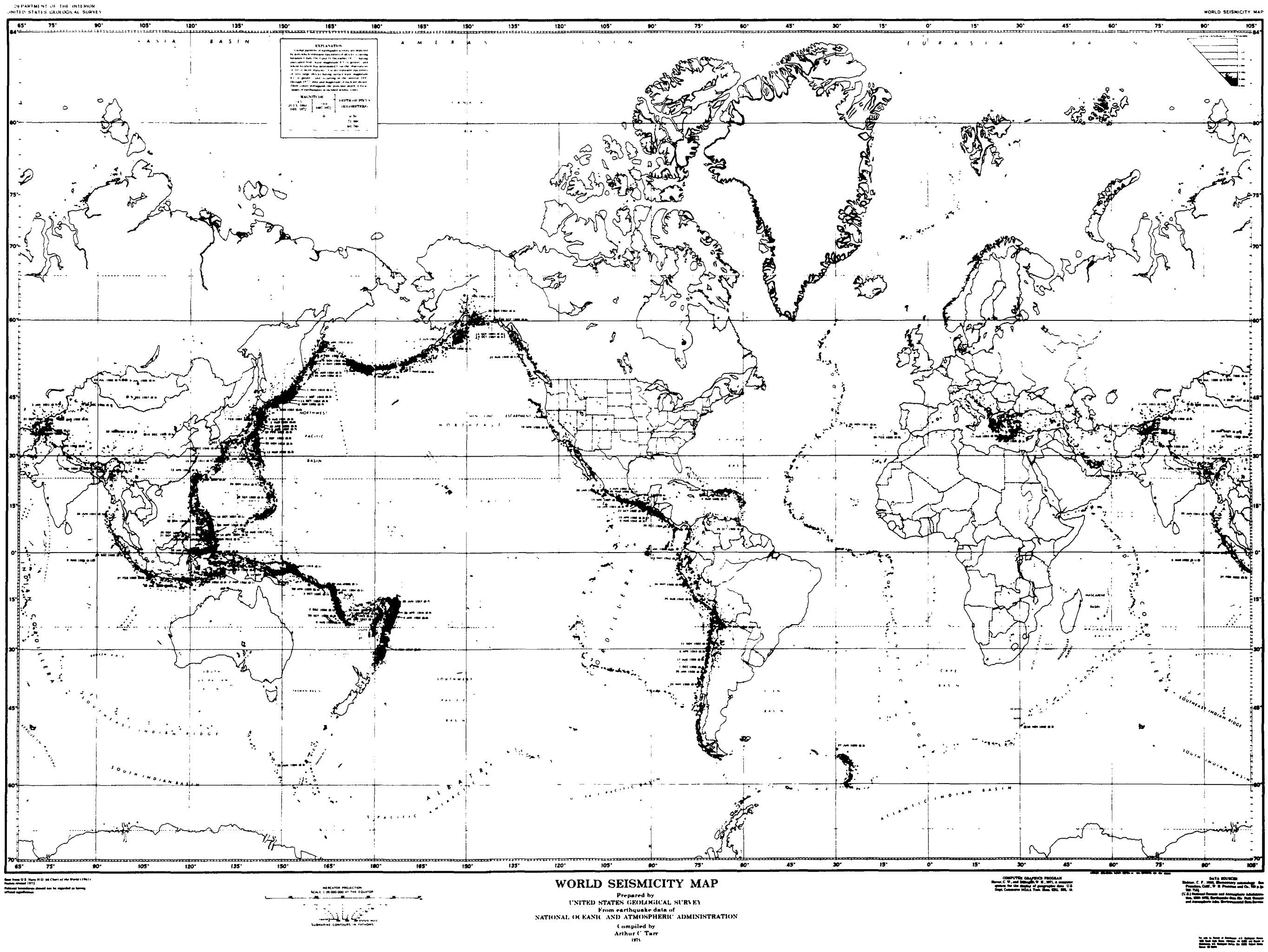

Fig. 1. World seismicity map. (U.S. Geological Survey. Compiled by A. C. Tarr, 1974.)

tive ones are shown in Fig. 7 for two horizontal and vertical directions. The traces for the upward motions are given above some of the horizontal traces. A good deal of useful information is obtained from such data as is shown in the next section.

GEOTECHNICAL IDEALIZATIONS

A simple schematic model of a mechanical oscillator consists of a mass attached to a thin vertical elastic rod fixed at the base. When such an oscillator is externally disturbed it vibrates with decreasing amplitude depending on the amount of system damping. A one-story building with the mass largely concentrated at the roof level approximates the suggested model of an oscillator. Disturbance of such a system can be caused by an earthquake ground motion. The differential equation for the horizontal motion of the roof for such a structure can be written as

$$\ddot{u} + 2\xi\omega\dot{u} + \omega^2 u = -\ddot{u}_g(t) \quad (1)$$

where $\ddot{u}$ is the acceleration of the roof, $\dot{u}$ its velocity, u its displacement, ξ the fraction of critical damping coefficient, or simply damping ratio, ω the natural circular frequency related to the natural period $T = 2\pi/\omega$, and $\ddot{u}_g(t)$ the ground acceleration such as given by an accelerogram.

Because earthquake ground accelerations are extremely irregular, analytical solutions of Eq. 1 are generally not possible, and numerical procedures are employed. For this purpose, after digitizing an accelerogram for a particular earthquake, and assuming a numerical value for T (or ω) and ξ for a structure, the response of such a structure can be calculated (Chopra, 1981; Clough and Penzien, 1975). Using this approach a complete history of response for a structure for a number of selected quantities such as deformation (deflection), velocity, and acceleration can be computed. The maxima of these quantities are of particular interest.

By repeating the above process for a great many values of T and a fixed value of ξ, plots of the response spectra for the maximum displacement, velocity, and acceleration can be obtained. These curves are simply the loci for the computed quantities. For any value of T on the x-axis, the corresponding ordinates give the maximum values of u, $\dot{u}$, or $\ddot{u}$ on plots. The response spectra for acceleration, i.e., for $\ddot{u}$, are favored in the present codes since conversion of such quantities into lateral forces follows by muliplying them by the mass of the structure.

Several acceleration response spectra for different strong earthquakes based on accelerograms near epicenters are shown in Fig. 8. All of these spectra have been generated from accelerograms digitized at 0.01 second intervals with the commmonly used damping ratio of 5%. The diversity in these spectra is striking. Note the large accelerations that develop for low period structures in several earthquakes (CH-Chile, LP-Loma Prieta, PA-Parkfield). The spectrum for the Mexico (MX) City earthquake, on soft alluvial soil and of long duration, is strikingly different from others. One of the earliest and best recorded, the El Centro (EC) earthquake, widely used in analyses, was not very violent; the Miyagi-Ken-Oki (MI) earthquake was also not particularly strong.

Currently accepted Applied Technology Council (ATC, 1978) idealizations for the acceleration spectra are superposed on the six spectral curves generated from earthquakes in Fig. 8. Mean spectral shapes based on 104 records, mostly in the western part of the U.S., are shown in Fig. 9 (Seed et al., 1976; NEHRP, 1988). Idealized normalized response spectra recommended for use in building design (NEHRP, 1988) are shown in Fig. 10. These spectra, being based essen-

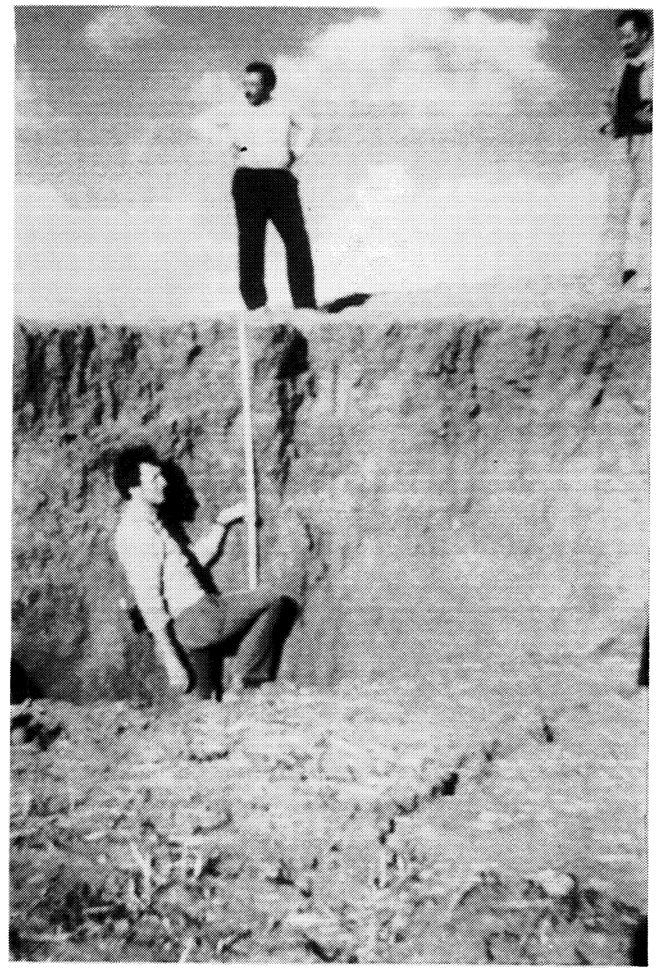

Fig. 3. Normal fault scarp near village Beni Rached during October 10, 1980 Algeria earthquake. (Courtesy of V. V. Bertero.)

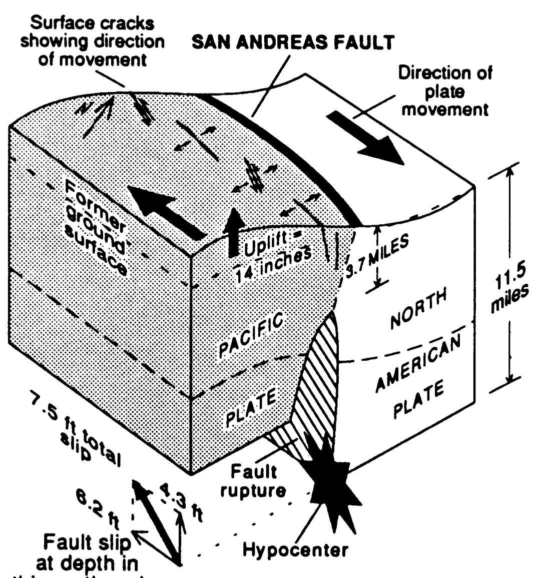

Fig. 2. Fault rupture mechanism for 1989 Loma Prieta earthquake. (U.S. Geological Survey. Modified from figure by M. J. Rymer.)

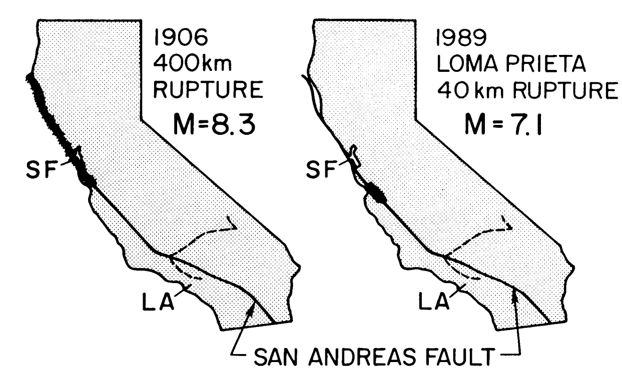

Fig. 4. Rupture lengths for two California earthquakes of different Richter intensity magnitudes along the San Andreas Fault.

tially on elastic concepts, will be referred to as linearly elastic response spectra, or, as elastic ultimate limit state spectra.

In accepting these spectra for design it is imperative to recognize specific site conditions. Great advances have been made by geologists, seismologists, and geotechnical engineers in identifying the effect of local site conditions on seismic risk. Microzonation of the land for man-made structures is assuming ever greater importance. For example, as early as 1975 the U.S. Geological Survey produced such a map for much of the San Francisco Bay Area. A fragment of such a microzonation map is shown in Fig. 11. The Marina District was clearly identified as a potentially hazardous area of the City of San Francisco. This conclusion was fully confirmed by the 1989 Loma Prieta Earthquake when many residential buildings in this district were severely damaged, although the epicenter of the quake was some 80 km (50 miles) away.

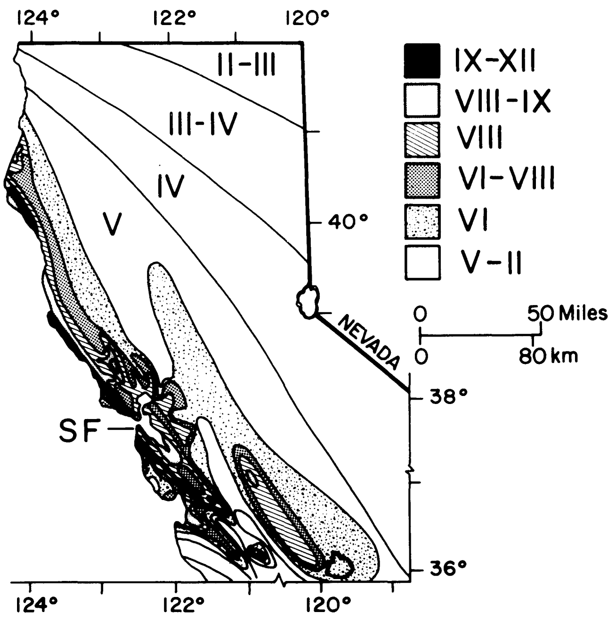

Fig. 5. Isoseismals for 1906 San Francisco earthquake using Rossi-Forel intensity scale varying from I to XII. (Courtesy B. Bolt.)

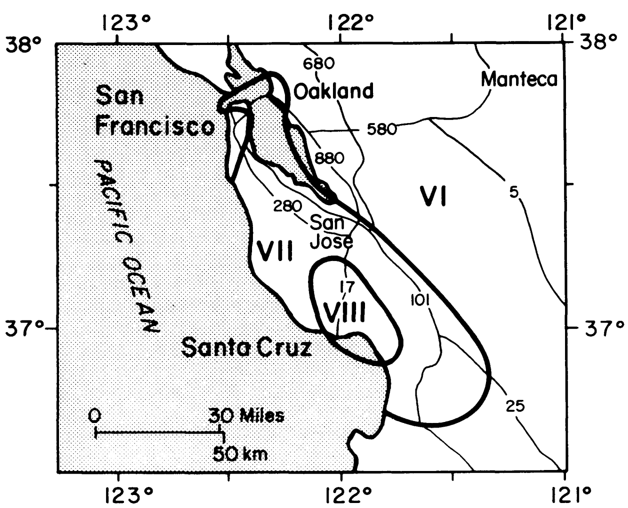

Fig. 6. Isoseismals for 1989 Loma Prieta earthquake using modified Mercalli intensity scale varying from I to X. (U.S. Geological Survey.)

SEISMIC STEEL CODE DEVELOPMENT

Providing minimum standards to assure public safety is the primary purpose of U.S. seismic codes. They are intended to safeguard against major failures and loss of life. Generally serviceability requirements during strong earthquakes

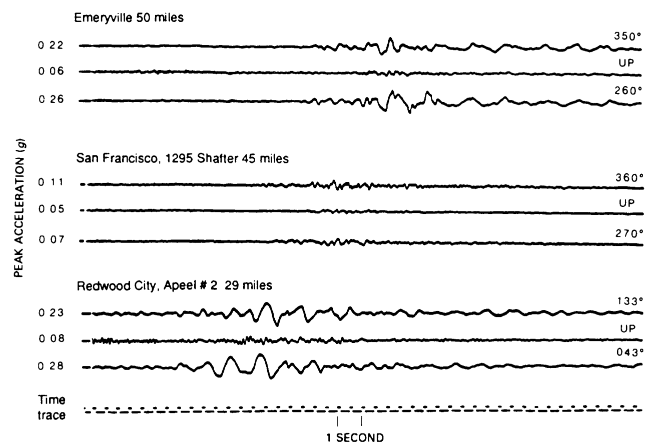

Fig. 7. Representative three-component accelerograms from 1989 Loma Prieta earthquake at various distances from epicenter at stations with different soil conditions. (U.S. Geological Survey.)

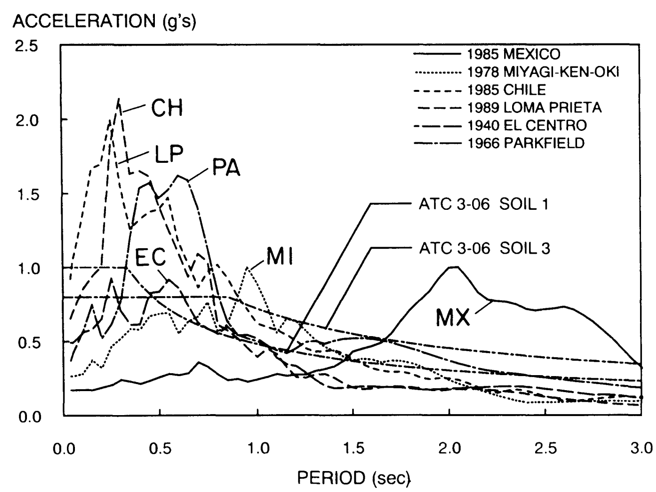

Fig. 8. Comparison of linear elastic response spectra with 5% damping for selected earthquakes. (Compiled by E. Miranda.)

are only implied but not required. In California, however, the construction of hospitals, public school buildings, fire stations, and a few other structures has added provisions for serviceability during strong earthquakes. Also, since 1972, special studies zones have been designated which limit construction near active fault traces. These limits primarily apply to new construction, and most existing residences are exempt from the state act. However only rudimentary codes are considered in this paper.

Historical Remarks

Development of codes for seismic-resistant construction has been evolving over a number of years and is applicable to all materials. California engineers and legislators were the initiators in the U.S. of the ever improving seismic codes. A schematic representation of the current situation and the interaction among several California and national groups is shown in Fig. 12.

San Francisco was rebuilt after the 1906 earthquake and fire using provisions of 30 psf (1.4 kPa) wind force. This was intended to safeguard construction from the effect of both wind and earthquakes. Only in 1927, and especially following the 1933 Long Beach Earthquake, was the concept of lateral earthquake forces proportional to mass firmly introduced into practice. In 1943 Los Angeles recognized the influence of the flexibility of a structure on earthquake design forces. San Francisco engineers beginning in 1948 culminated their efforts in 1952 in developing a relationship stating that seismic forces are inversely proportional to the period T of a structure.

In 1959 the Seismology Committee of the Structural Engineers Association of California (SEAOC) provided the first Recommendations relating the type of framing system to the anticipated period of a structure. Thereafter continuous improvements were made by SEAOC, and, after scrutiny, generally adopted by UBC. Because of the magnitude of the task in revising the code, SEAOC created ATC, which subsequently became an independent national organization. Its monumental task was the completion, in 1978, of the ATC-3-06 document on the Tentative Provisions for the Develop-

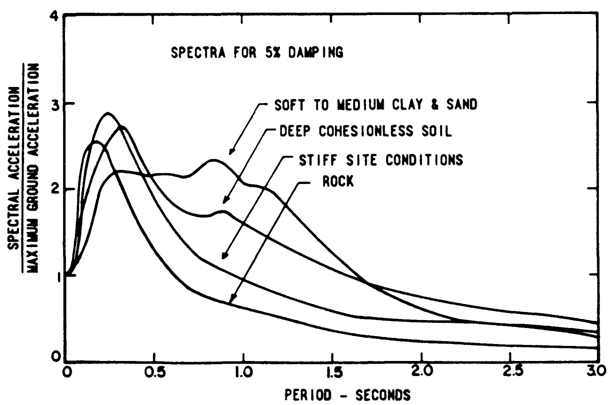

Fig. 9. Average acceleration spectra for different site conditions. (After Seed, et al., 1976; NEHRP, 1988.)

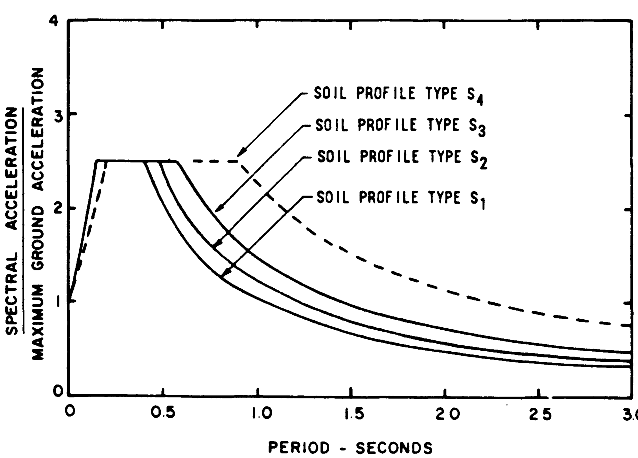

Fig. 10. Normalized response spectra recommended by 1988 NEHRP for use in building codes. (NEHRP, 1988.)

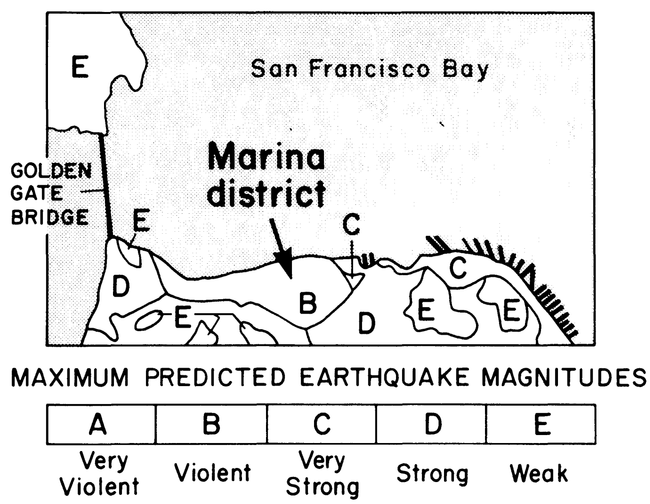

Fig. 11. 1975 U.S. Geological Survey microzonation map fragment for San Francisco showing remarkable correlation with observations after 1989 Loma Prieta earthquake.

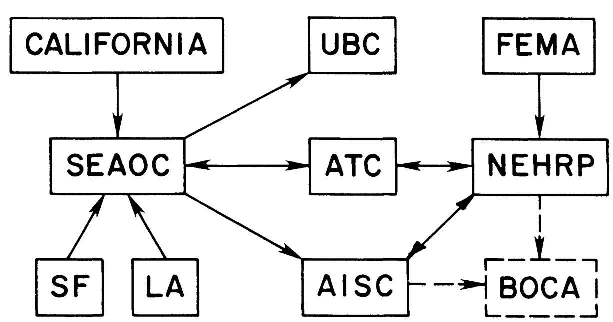

Fig. 12. Evolution of U.S. seismic codes.

ment of Seismic Regulations for Buildings. In its present form this updated document is promulgated as the 1988 NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings.

In the intervening years SEAOC heavily contributed to the continual updating of UBC. Moreover, after a full decade of working on a complete overhaul of the seismic code, SEAOC Recommended Lateral Force Requirement and Commentary was published in 1990. This was preceded in 1985 by their issuance of Tentative Lateral Force Requirements, incorporating the revised provision for a new code. This overall effort involved thousands of volunteer hours. The code itself, with a few changes, became a major part of the 1988 UBC Structural code.

The latest newcomer to the realm of seismic steel codes is the 1990 AISC Load & Resistance Factor Design (LRFD) Seismic Specifications. AISC is publishing this document as a stand-alone publication in its first issue. It is intended to become a part of the 1986 AISC LRFD Specifications. Many of the provisions in this new seismic code are similar to the 1990 SEAOC Recommendations and the 1988 UBC. However the lateral load requirements are stipulated on the basis of the 1982 ANSI (American National Standards Institute) A58.1, which now has been assumed by the American Society of Civil Engineers (ASCE) and will be referred to in this paper as the 1990 ASCE 7-88 Recommendations. These recommendations are in the process of being revised. Nevertheless, since this is the reference document for the 1986 AISC LRFD Specifications, it is retained in the new AISC seismic provisions. However, the 1990 AISC LRFD Seismic Provisions are so written that any other appropriate seismic lateral loads can be used.

To summarize, at present in the U.S. in the public domain there are four documents for the seismic design of steel buildings: the 1988 UBC, the 1990 AISC LRFD Seismic Specifications, the 1988 NEHRP, and the 1990 SEAOC Recommendations. Except for UBC, the remaining three documents have extensive commentaries. Repeated reference is made to these four documents in the subsequent discussion. It is of interest to note that BOCA (see Fig. 12) is in the process of including seismic provisions.

Basic Seismic Lateral Load Provisions

In this section it will be shown that the basic seismic lateral loads currently specified in relevant U.S. steel codes and recommendations, although stated in different terms, are fundamentally very much alike. It is convenient to subdivide these lateral load provisions into three groups. The 1988 UBC and the 1990 SEAOC Recommendations are essentially the same, and both are formulated for use for allowable stress design (ASD). The 1988 NEHRP Recommendations stipulate the first significant yield as the strength criterion. The 1990 AISC LRFD is also based on the first significant yield, but contains reliability considerations requiring the use of load and resistance factors. These provisions are based on the 1990 ASCE 7-88 document, as does, in essence, the main body of the 1986 AISC LRFD specifications.

It is important to know that the empirical inelastic design response spectra are obtained from the linear elastic response spectra. Either the 1988 UBC or the 1988 NEHRP provisions can be used for illustration. The 1990 AISC Specifications, being based on the 1988 ASCE 7-88, are unsuitable since the inelastic design response spectra are given directly. The more fundamental approach by the 1978 ATC/1988 NEHRP, and later adopted by the 1990 SEAOC/1988 UBC, is in the process of being included into the forthcoming updated ASCE 7-88 document.

To illustrate the spectral transformation consider a building requiring compliance with the 1988 UBC. Assume that the building is in Seismic Zone 4, having the seismic zone factor $Z = 0.4$, and a standard occupancy importance factor $I = 1$. Then assume further that the soil profile has stiff soil conditions requiring $S_2 = 1.2$. The elastic ultimate state spectrum for the conditions stated, shown in Fig. 13, is based on the curve given in Fig. 10 for soil S_2. (The equations defining such curves are discussed in greater detail later.) In this plot the ordinates are given as the ratio of a building's seismic base shear V_B to its weight W as a function of the building's period T.

The upper curve representing the linearly elastic response spectrum, or the ultimate limit state spectrum as it is referred to earlier, represents the best consensus information available from geotechnical engineers. For major projects, after a thorough geotechnical investigation, the code specified spectrum is often replaced by a more appropriate one for the site conditions. After a spectral curve is selected, its ordinates are greatly reduced by coefficients, such as $1/R_w$ in the 1988 UBC, in order to obtain the inelastic design response spectrum. Similar reduction coefficients are used in the 1988 NEHRP, as well as in the 1990 ASCE 7-88/1990 AISC LRFD.

It is to be noted that when the 1990 ASCE 7-88 equation with different K's is used, the inelastic design response spectra are obtained directly. However, applying either the 1988 UBC, or the 1988 NEHRP, the inelastic design response spectra are determined from an elastic response spectra by dividing, respectively, by an appropriate R_w or R factor. Using this second more fundamental approach a finer selection of reduction factors can be made to correspond to different structural systems.

Following the second approach the ordinates C_{eu} of the elastic ultimate limit state spectrum are divided by $R_w = 8$ to yield an UBC inelastic design response spectrum, shown in Fig. 13 by the solid line, for multiple degree-of-freedom systems. This dramatic decrease in the values of the ordinates is applicable for fairly ductile structural systems. For some steel structures R_w can be increased to 12, resulting in the spectrum shown in the figure by dashed lines.

It is to be noted that R_w's are constant throughout the whole range of structure periods T. Therefore there is no ground-motion period dependence. Moreover, as noted by Rojahn and Hart (1989), R_w's are based on committee consensus, and do not have adequate analytical support nor field data. Acceptable damage is not defined. The use of inelastic design response spectra deduced from an elastic single degree of freedom to multiple degrees of freedom systems is not adequately explored. Some investigators (Uang and Bertero, 1988) question the adequacy of a single parameter. The base shear V_B, determined from spectral acceleration, is not sufficiently general to include the necessary parameters for destructive earthquakes. These investigations show that the more reliable parameter for defining earthquake damage potential is earthquake energy input. In this regard the duration of an earthquake plays a very important role. These and other issues pertaining to earthquakes require further intensive research.

Comparisons Among Code Design Spectra

In spite of criticism raised regarding the inelastic design response spectra, they are the basis for current design. Therefore it is instructive to make a comparison between the three alternative formulations currently available. These are (1) the 1990 ASCE 7-88/1990 AISC LRFD, (2) the 1988 UBC/1990 SEAOC, and (3) the 1988 NEHRP. The respective inelastic design response spectra for the three cases are defined as follows:

1990 ASCE 7-88/1990 AISC LRFD:

$$E \equiv V_B = ZIKSCW \tag{2}$$

where $C = 1/15\sqrt{T} \le 0.12$ and $CS = 0.14$ maximum.

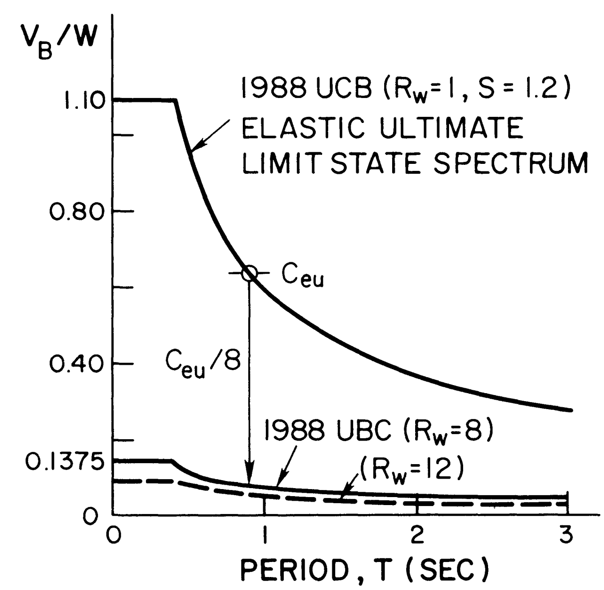

Fig. 13. Empirical seismic response spectra.

In this equation $E \equiv V_B$ = Base shear, Z = Seismic zone factor, varying from $3/16$ to a maximum of 1; I = Importance factor, varying between 1.0 and 1.5; K = Factor dependent on the structural framing system, varying from a minimum of 0.67 to 1.33; S = Soil factor; C = Seismic coefficient dependent on period T; W = Total dead and partial live load.

1988 UBC/1990 SEAOC:

$$E \equiv V_B = ZICW / R_w \tag{3}$$

where $C = 1.25S/T^{2/3} \le 2.75$

In this equation the new symbols and those with different definitions are: Z = Seismic zone factor, varying from 0.075 to 0.40; C = Soil and period dependent seismic coefficient; R_w = Factor dependent on the structural framing system, varying from 4 to 12. This equation with $Z = 0.40$, $I = 1$, $S = 1.2$, and $R_w = 1$ is used to define the elastic limit state spectrum in Fig. 13.

1988 NEHRP:

$$E \equiv V_B = C_s W \tag{4}$$

where $C_s = 1.2A_v S / RT^{2/3} \le 2.5A_a / R$

In this equation the new symbols are: R = Response modification coefficient dependent on framing system, varying from 2 to 8; A_v = Seismic coefficient for velocity-related acceleration with a maximum of 0.40; A_a = Seismic coefficient for effective peak acceleration with a maximum of 0.40.

For comparison it is convenient to recast the above three equations to express the base shear coefficients V_B/W.

1990 ASCE 7-88/1990 AISC LRFD:

$$V_B/W = ZIKS / 15\sqrt{T} \tag{5}$$

and if $Z = 1$, $I = 1$, $K = 1$, and $S = 1.2$,

$$V_B/W = 0.80 / \sqrt{T} \le 0.14 \tag{6}$$

1988 UBC/1990 SEAOC:

$$V_B/W = 1.25ZIS/R_w T^{2/3} \tag{7}$$

and if $Z = 0.40$, $I = 1$, $S = 1.2$, and $R_w = 8$, which corresponds to $K = 1$ in Eq. 5,

$$V_B/W = 0.075 / T^{2/3} \le 0.1375 \tag{8}$$

1988 NEHRP:

$$V_B/W = 1.2A_v S/RT^{2/3} \tag{9}$$

In order to obtain from this relation a reduced equation comparable to Eqs. 6 and 8 let $A_v = A_a = 0.4$, $S = 1.2$, $R = 5$, which approximately corresponds to $K = 1$ or $R_w = 8$. Since, however, the preceding two formulations are based on the ASD, whereas in the 1988 NEHRP the first significant yield is used as the strength criterion, Equation 9 must be divided by the ratio of yield stress to an allowable stress. Taking this ratio as 36 / 24 = 1.5, the reduced equation is

$$V_B/W = 0.077 / T^{2/3} \le 0.20 / 1.5 = 0.1333 \quad (10)$$

The plots of Eqs. 6, 8, and 10 are shown in Fig. 14. For comparison purposes, the limiting values for these curves, such as 1988 UBC's 0.1375 are given to a larger number of significant figures than warranted by the supporting data.

Note the heights of the 1988 UBC curves shown in Figs. 13 and 14. The ordinates for the 1988 UBC curve shown in Fig. 14 are drawn to a much larger scale than those in Fig. 13, and the comparisons of the three design spectral curves is made at this scale.

The agreement among the three sets of curves shown in Fig. 14 is reassuring, but since K's, R_w's, and R's are based on consensus, their true accuracy can be questioned. Several investigators have pointed out, for example, that in the low period range the design spectra may be deficient (Bertero and Bresler, 1977). The author shares this opinion. See for example the spectra in Fig. 8 in the low range of periods for the Loma Prieta, Chile, and Parkfield earthquakes.

In comparing the reduced 1988 NEHRP design rcsponse spectra it is important to keep in mind that the 1990 ASCE 7-88 and the 1988 UBC spectra are intended for use with ASD. However, in the 1990 AISC LRFD Seismic Specifications, based on the 1990 ASCE 7-88 Recommendations, the basic seismic load factor is 1.5. Therefore on multiplying the 1990 ASCE 7-88 spectra by 1.5, the equivalence to the 1988 NEHRP unreduced spectra formulated for yield is obtained.

Amplified Earthquake Load Provisions

In the U.S. structural steel codes some members and connections are required to be designed for substantially larger seismic lateral loads than the ones discussed in the previous section. Such amplified earthquake loads are required to be considered in the design of columns, beam-to-column joints, bracing connections, and for determining story drifts.

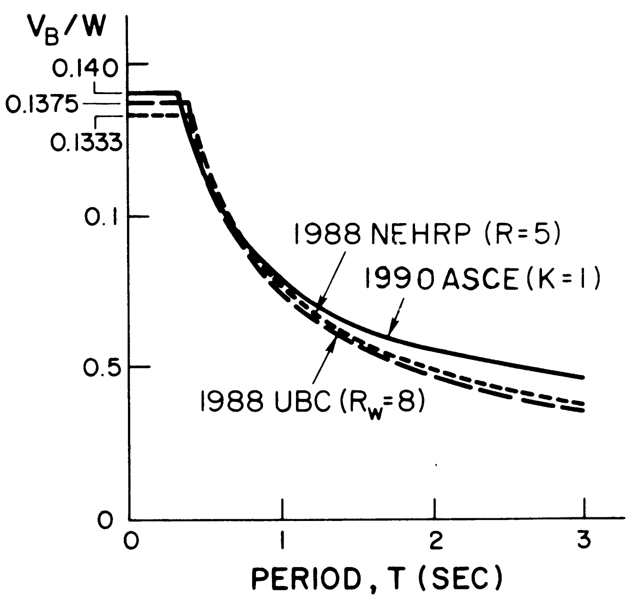

Fig. 14. Inelastic design response spectra for soil S = 1.2 by three codes. For hard rock: S = 1 (÷ 1.2); for soft soil S = 1.5 (× 1.25); for very soft soil S = 2 (× 1.67).

The amplified earthquake loads for the three cases considered in this paper are obtained using different amplification factors. Thus for the 1990 ASCE 7-88/1990 AISC LRFD the amplification factor is $3/K$ (proposed by Pinkham), for the 1988 UBC it is $3R_w/8$, and for the 1988 NEHRP at the allowable stress level it is $2R/5$. Applying these factors to Equations 2, 3, and 4, the amplified earthquake loads V_B' (or E') become:

1990 ASCE 7-88/1990 AISC LRFD:

$$V_B' = 3V_B/K = 3ZISCW \le 0.42W \quad (11)$$

1988 UBC/1990 SEAOC:

$$V_B' = (3R_w/8)V_B = \tfrac{3}{8}ZICW \le 0.41W \quad (12)$$

1988 NEHRP:

$$V_B'(2R/5)V_B = 0.48A_vSW/T^{2/3} \le 0.40W \quad (13)$$

The plot of these three functions divided by W, except for the substantially larger values of the ordinates, completely resembles Fig. 14. It is such a plot as this that is presented in the 1990 AISC LRFD Commentary for Seismic Provisions. Note that for these amplified earthquake design loads there is no dependence on the type of structural framing system as K's, R_w's, and R's cancel out in algebraic manipulations. This lack of influence of these parameters on maximum lateral forces is a debatable issue.

Code Design and Structural Response

Linear elastic analyses are used in conventional seismic design in the U.S. For large loads this procedure cannot provide either the true strength of members and connections nor the inelastic displacement of a structure. However designers clearly appreciate the difference between brittle and ductile behavior of structures during severe earthquakes. A schematic illustration of the behavior of structures to failure for a monotonically increasing load is shown in Fig. 15. Elastic analyses offer no information on the magnitudes of the ultimate load or deformation.

The codes, recognizing the preceding problems as ductility related, specify smaller reduction factors for brittle than for ductile structural systems. For example, for a properly detailed steel moment resisting frame $R_w = 12$ (see Fig. 13), whereas for a brittle system R_w may be as small as 4. The justification for the ductilities of the systems are consensus opinions based on performances during past earthquakes and laboratory tests.

The burden of proof for the ductility of a system is generally placed on tests of members and connections. This includes their behavior under severe cyclic loads. The system ductility, idealized in Fig. 15 by elastic-plastic behavior, is larger than that corresponding to the formation of the first plastic hinge. However this presupposes that the critical members (and connections) can sustain the required ductility during the whole inelastic deformation process. Unless the system is optimized, distress in a well designed ductile structure occurs only after a considerable amount of plastic action at several plastic hinges. A meaningful yield level for the ductile system shown in Fig. 15 occurs at the level of V_{y-2}. This system yield is usually considerably higher than that occurring at the first plastic hinge. The ratio of system yield to first plastic hinge yield is the overstrength factor. Emphasized early by Blume (1977), it is receiving renewed attention (Uang and Bertero, 1988; Rojahn, 1988; Uang, 1991). For some steel frames Uang and Bertero, as well as Whittaker, et al. (1987), estimated this factor to be over 2. This overstrength is due to the statical indeterminancy of a system, selection of oversize members to control drift, statistically superior material strength, etc. For optimized structural systems, though, the overstrength factor may be smaller.

The current seismic design code procedure is schematically illustrated in Fig. 16. The 1988 UBC/1990 SEAOC approach utilizes the ASD. The 1978 ATC/1988 NEHRP is an ultimate strength design based on the formation of the first plastic hinge. The 1990 AISC LRFD provisions enhance the 1988 NEHRP approach with load and resistance factors. Note that the reduction factor R for the 1988 NEHRP is smaller than R_w for 1988 UBC. Both of these factors are larger than R_y, the system reduction factor. The overstrength factor Ω for the two methods of calculation can be defined as follows:

$$\Omega_{UBC} = R_w/R_y \tag{14}$$

$$\Omega_{NEHRP} = R/R_y \tag{15}$$

No quantification of these large factors is generally available at the present time.

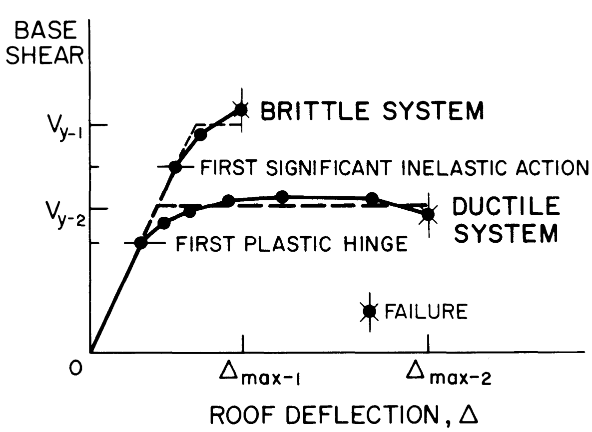

Fig. 15. Elastic-plastic idealizations of brittle and ductile systems.

FUTURE RESEARCH CHALLENGES

During the past several years great progress has been made in improving seismic codes. Understanding of the very difficult seismic problem has also been greatly enhanced. There is better interaction now among seismologists, geotechnical engineers, and structural engineers. However many problems remain. The foremost unresolved problem may be the use in design of a single parameter based on acceleration. Such a parameter completely neglects the duration of an earthquake, known to be very important. An energy based criterion may be able to provide a quantum advance in seismic design. A more precise definition of acceptable damage is also required.

Academia should be able to provide required simplified computer procedures for nonlinear structural analyses. The 1978 ATC/1988 NEHRP use of material yield rather than the ASD is a step forward. An enhancement of this approach by AISC in going to LRFD is commendable.

Re-examination in the curent codes of the reduction factors R's and R_w's requires immediate attention. These are now largely based on consensus. The use of such constant reduction factors irrespective of the period of the structure is questionable. These factors are probably too large, particularly in the low period range. The contribution of the overstrength of structural systems to the reduction factors should be clearly recognized and quantified. The likely reduction in the overstrength factors in optimum design requires attention.

Requirement for a two-level design for serviceability and ultimate strength would be a welcome addition to U.S. codes.

Further studies of nonlinear behavior of buildings with multiple degrees-of-freedom, especially those with irregularities, should continue.

The attractive possibilities of energy dissipative and base isolated systems should be further explored.

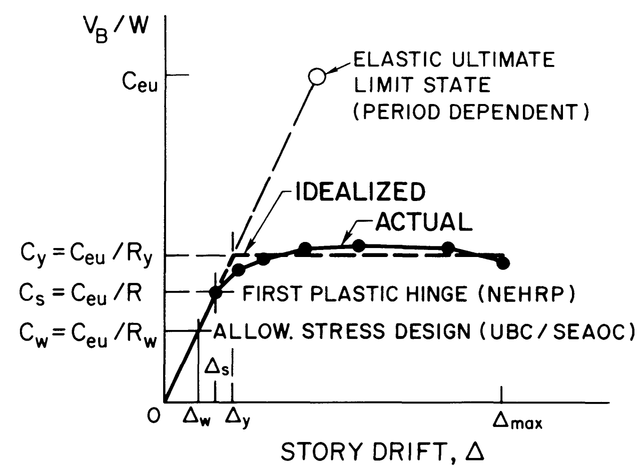

Fig. 16. Structural system response. (Adopted from Newmark/Hall, 1982; Uang/Bertero, 1988; Uang, 1991.)

ACKNOWLEDGMENTS

The support of the National Science Foundation (current Grant No. ECE-8603320) is gratefully acknowledged. The continued encouragement of Dr. S. C. Liu of the NSF is much appreciated. Continuous service, since its inception, as Chairman of the AISC LRFD Subcommittee on Seismic Provisions has been most rewarding, and particular gratitude is expressed to C. W. Pinkham, Secretary of this committee. The assistance of graduate student Carl Grigorian with the final editing of the paper is also gratefully acknowledged.

REFERENCES

AISC LRFD, 1986. *Load and Resistance Factor Design (LRFD) for Structural Steel Buildings,* American Institute of Steel Construction, Chicago, Illinois.

AISC LRFD, 1990. *Seismic Provisions,* AISC, Chicago, Illinois.

ANSI, 1982. *Building Code Requirements for Minimum Design Loads in Buildings and Other Structures,* American National Standards Institute Report ANSI A58.1, New York.

ASCE, 1990. *Minimun Design Loads for Buildings and Other Structures,* American Society of Civil Engineers ASCE 7-88, New York, NY.

ATC-3-06, 1978. *Tentative Provisions for the Development of Seismic Regulations for Buildings,* Applied Technology Council, National Bureau of Standards, Washington, D.C.

Bertero, V. V., and Bresler, B., 1977. *Failure Criteria (Limit States),* Proc. 6th World Conf. Earthquake Engrg., New Delhi, pp. 77–87.

Blume, J. A., 1977. *Allowable Stresses and Earthquake Performance,* Proc. 6th World Conf. Earthquake Engrg., New Delhi, pp. 165–174.

Calder, N., 1972. *The Restless Earth.* The Viking Press, New York.

Chopra, A. K., 1981. *Dynamics of Structures—A Primer,* Earthquake Engineering Research Institute, Berkeley, California

Clough, R. W., and J. Penzien, 1975. *Dynamics of Structures,* McGraw-Hill, New York.

NEHRP, 1988. *Recommended Provisions for the Development of Seismic Regulations for New Buildings (Parts 1 and 2),* Federal Emergency Management Agency (FEMA), Building Seismic Safety Council, Washington, D.C.

Plafker, G., and Galloway, J. P., 1989. *Lessons Learned from the Loma Prieta, California, Earthquake of October 17, 1989,* U.S. Geological Survey Circular 1045, Department of Interior, Washington, D.C.

Rojahn, C., 1988. *An Investigation of Structural Response Modification Factors,* Proc. 9th World Conf. Earthquake Engrg., Vol. V, Tokyo, pp. 1087–1092.

Rojahn, C., and G. Hart, 1989. *U.S. Code Focusing on R-Factor of UBC, ATC-3 and NEHRP, Third U.S.-Japan Workshop on the Improvement of Building Structural Design and Construction Practices,* ATC 15-2, Applied Technology Council, Redwood City, California.

SEAOC, 1959, 1985 (Tentative), 1990. *Recommended Lateral Force Requirements and Commentary,* Seismology Committee, Structural Engineers Association of California, Los Angeles/Sacramento/San Francisco.

Seed, H. B., C. Urgas, J. Lysmer, 1976. "Site Dependent Spectra for Earthquake-Resistant Design," *Bulletin Seism. Soc. of America,* 66, (1):221–244.

Uang, C-M., 1991 (in press). "On Establishing R (or R_w) and C_d Factors for Seismic Provisions," *J. Struct. Engrg.,* ASCE, Vol. 117, January 1991.

Uang, C-M., and V. V. Bertero, 1988. *Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Concentrically Braced Steel Structure,* Report UCB/EERC-86/10, Earthquake Engineering Research Center, University of California, Berkeley.

UBC, 1988. *Uniform Building Code,* International Conference of Building Officials, Whittier, California.

Whittaker, A. S., C-M. Uang, and V. V. Bertero, 1987. *Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Eccentrically Braced Steel Structure,* Report UCB/EERC-87/02, Earthquake Engineering Research Center, University of California, Berkeley.