An Overview of Seismic Codes

Engineers should be familiar with basic seismic concepts, the development of current seismic codes and the possible changes that might be made to the codes in the near future.

JAMES ROBERT HARRIS

The last five years have been a time of substantial change in the treatment of earthquake safety in building codes in the United States. It is anticipated that the amount of work on seismic codes will not diminish in this decade. It is important that engineers have a firm understanding of:

- The fundamentals of seismic demand and response
- The historical development of fundamentals within U.S. codes
- The present status and future directions of U.S. codes

In introducing their well-known text, *Fundamentals of Earthquake Engineering*, Newmark and Rosenblueth comment:

"In dealing with earthquakes, we must contend with appreciable probabilities that failure will occur in the near future. Otherwise, all the wealth of the world would prove insufficient to fill our needs: the most modest structures would be fortresses. We must also face uncertainty on a large scale, for it is our task to design engineering systems — about whose pertinent properties we know little — to resist future earthquakes and tidal waves — about whose characteristics we know even less. . . . In a way, earthquake engineering is a cartoon. . . . Earthquake effects on structures systematically bring out the mistakes made in design and construction, even the minutest mistakes."[1]

There are several points that are essential to understanding the theories and practices of earthquake-resistant design:

- Ordinarily, a large earthquake produces the most severe loading that a building is expected to survive. The probability that "failure" will occur is very real and is greater than for other loading phenomena. Also, in the case of earthquakes, the definition of "failure" is altered to permit certain types of behavior that are considered failure in relation to the effects of other phenomena.
- The levels of uncertainty are much greater than those encountered in the design of structures to resist other phenomena. This

uncertainty applies both to knowledge of the loading function and to the resistance properties of the materials, members and systems.

- The details of construction are very important because flaws of no apparent consequence often will cause systematic failure simply because the earthquake loading is so severe and because an extended range of behavior is permitted.

A key text, the *National Earthquake Hazard Reduction Program (NEHRP) Recommended Provisions for the Development of Seismic Regulations for Buildings*, presents one set of concepts on which earthquake-resistant building design may be based. Produced by the Building Seismic Safety Council (BSSC) and sponsored by the Federal Emergency Management Agency (FEMA), this text is composed of two volumes and a set of maps. Part 1 contains the actual provisions (referred hereafter as the *Provisions).* Part 2 provides the commentary explaining the various aspects of the provisions (referred hereafter as the *Commentary).* These provisions are the source of new seismic provisions being introduced into building codes and standards in most of the East, South, and Midwest. They are technically very similar to those used in the West since 1988.

Earthquake Phenomena

According to the most widely held scientific belief, most earthquakes occur when two segments of the earth's crust suddenly move in relation to one another. The surface along which movement occurs is known as a fault. The sudden movement releases energy and causes seismic waves to propagate through the crust surrounding the fault. These waves cause the surface of the ground to shake violently. This ground shaking is the principal concern of building design.

This type of earthquake is *tectonic* in origin. Nearly all large earthquakes are associated with movements of, and strains in, large segments of the earth's crust or "plates." Virtually all such earthquakes occur at or near the boundaries of these plates. Tectonic earthquakes occur in the far western portion of the United States where two very large plates — the North American continent and the Pacific basin — come together. Abrupt ground displacements occur where a fault intersects the ground surface, commonly occurring in California earthquakes.

In the central and eastern United States, however, earthquakes (such as the historic Charleston, South Carolina, earthquake or the very large New Madrid, Missouri, earthquakes of the previous century) are not associated with such a plate boundary, and their causes are not as completely understood. This factor, combined with the smaller amount of data concerning central and eastern earthquakes (because of their infrequency) means that the uncertainty associated with earthquake loadings is higher in the central and eastern portions of the country than in the West.

In tectonic earthquakes, the amplitude of earthquake ground shaking diminishes with distance from the source. In addition, the rate of attenuation varies for different frequencies of motion, being less for low frequencies than high frequencies. Therefore, the *Provisions* includes two maps for seismic hazard zoning. One map reflects the quicker attenuation for higher frequency motion (the A_a map) and the other reflects the slower attenuation for lower frequencies (the A_v map).

Two basic data sources are used in establishing the likelihood of earthquake ground shaking, or seismicity, at a given location. The first is the historical record of earthquake effects and the second is the geological record of earthquake effects. Given the infrequent occurrence of major earthquakes, there is no place in the United States where the historical record is long enough to be used as a reliable basis for earthquake prediction. Even on the eastern seaboard, the historical record is too short to justify sole reliance on the historical record. Therefore, the geological record is essential. Such data require very careful interpretation, but they are widely used to improve our knowledge of seismicity.

While geological data have been developed for many locations throughout the country as part of the nuclear power plant design process, there are more geological data available, on the whole, for the far western United States than for any other region. Both sets of data have been taken into account in the seismic hazard maps

included in the *Provisions*. However, ground shaking has been shown to vary considerably over small distances and that the maps in the *Provisions* do not attempt to show such small-scale variations, commonly called microzoning.

The *Commentary* provides a more thorough discussion of the development of the seismic hazard maps, their probabilistic basis, the necessarily crude lumping of parameters, and so forth. Although the 1988 edition of the *Provisions* contains an "Appendix to Chapter 1" that provides alternate maps and methods for establishing design ground motion, the 1991 edition does not contain this appendix. In its place there is a new set of maps and a new procedure to develop a design response. The *Commentary* discussion provides insight into the perceived need for, and use of, these alternative maps and methods. The maps with longer exposure periods are intended to allow users to develop some perspective on the issue of performance should a rare event occur. The entire concept for establishing design ground motions is being debated by the BSSC committees currently updating the *Provisions* and it is likely that design spectra (formulae for establishing base shear) will be revised in future editions.

Mass soil failures such as landslides, liquefaction and gross settlement are the result of ground shaking on susceptible soil formations. Once again, design for such events is very specialized and it is common to locate buildings so that mass soil failures and fault breakage do not pose major consequences to their performance. Modification of soil properties to protect against liquefaction is one important exception. The structural loads specified in the *Provisions* are based solely on ground shaking; they do not provide for ground failure.

Earthquakes have many other effects in addition to ground shaking. However, most of these other effects do not generally become major considerations in building design for various reasons. For example, seismic sea waves, or tsunamis, can cause very forceful flood waves in coastal regions, and seiches (long-period "sloshing") of lakes and inland seas can have similar effects along shorelines. While these effects are outside the scope of the *Provisions*, they should be given consideration during site exploration and analysis. The design of structures to resist such hydrodynamic forces, however, is a very specialized topic, and it is common practice to avoid constructing buildings where such phenomena are likely to occur.

Response of Buildings to Ground Shaking

The important difference between building response to an earthquake and its response to most other loadings is that the response to an earthquake is *dynamic*, not *static*. For most buildings, even the response to wind is essentially static. Although wind pressures are variable, responsive building motions are so slow in relation to the ordinary frequencies of structural vibration that stresses within the structure are due almost entirely to the pressure loading rather than acceleration of the mass of the structure. However, with earthquake ground shaking, the aboveground portion of a building is not subjected to any applied force. The stresses and strains within the superstructure are created entirely by its dynamic response to the movement of its base, the ground. Even though the most used design procedure resorts to the use of a concept called the equivalent static force for actual calculations, some knowledge of the theory of vibrations of structures is essential.

Response Spectra. Figure 1 presents accelerograms (records of the acceleration at one point along one axis) for several representative earthquakes. Great earthquakes can extend for much longer periods of time than the 30 seconds depicted in the figure. For example, the 1964 Alaska earthquake exhibited strong shaking for over three minutes. Note the random nature of the ground shaking and the range of characteristics of the different accelerograms. The precise analysis of the elastic response of an ideal structure to such a pattern of ground motion is possible; however, it is not commonly done for building structures. Perhaps the increasing power and declining cost of computational aids will make such analyses more common, but at this time only exceptional structures are analyzed for specific response to a specific ground motion.

Figure 2 shows further detail developed from an accelerogram for a seven-story rein-

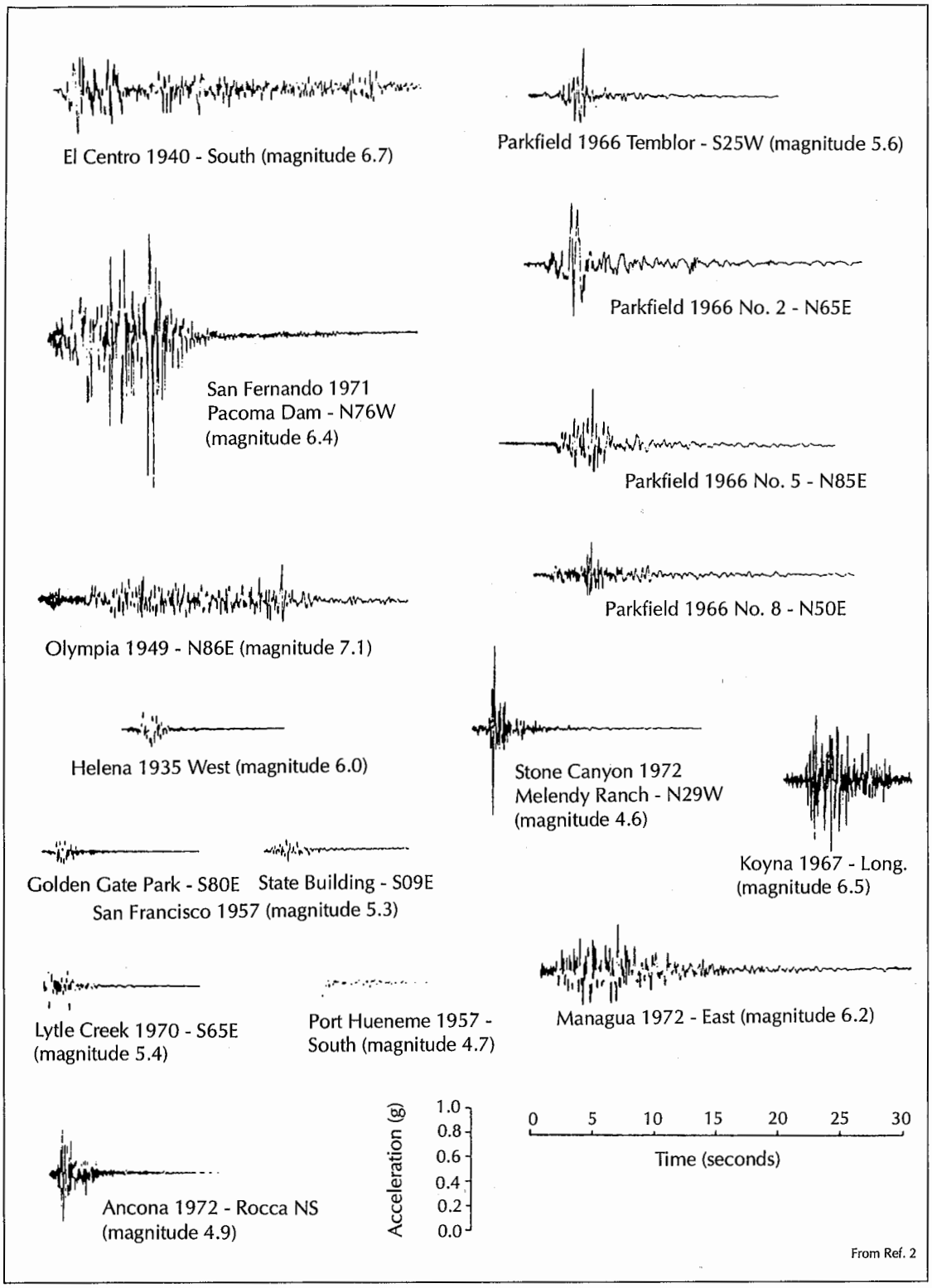


FIGURE 1. Earthquake ground acceleration in epicentral regions. All accelerograms are plotted to the same scale for time and acceleration.

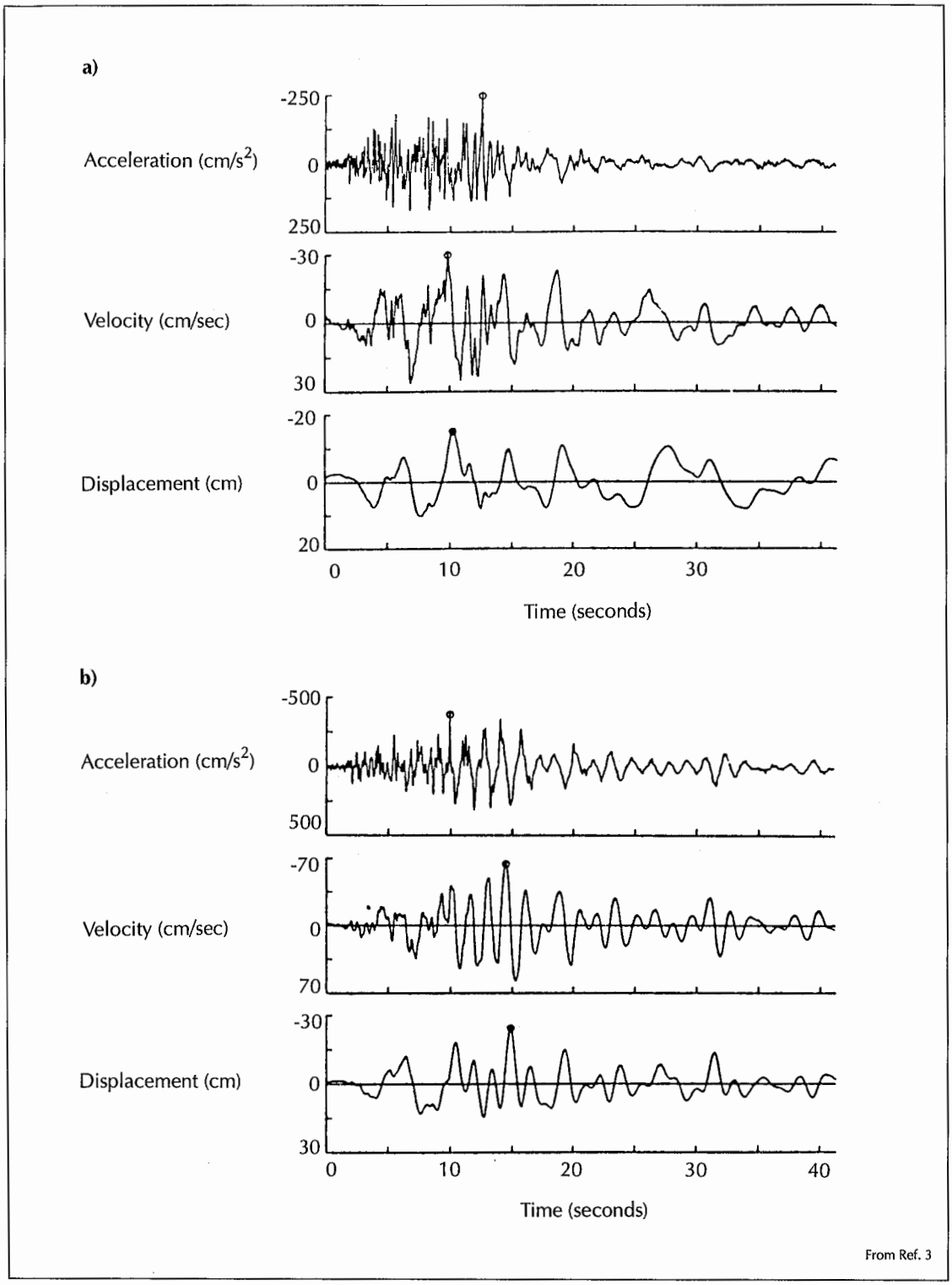


FIGURE 2. Accelerogram with computed velocity and displacement for a seven-story building during the magnitude 6.4 San Fernando Earthquake in 1971. The north-south ground motion is shown in (a); the north-south roof motion is depicted in (b).

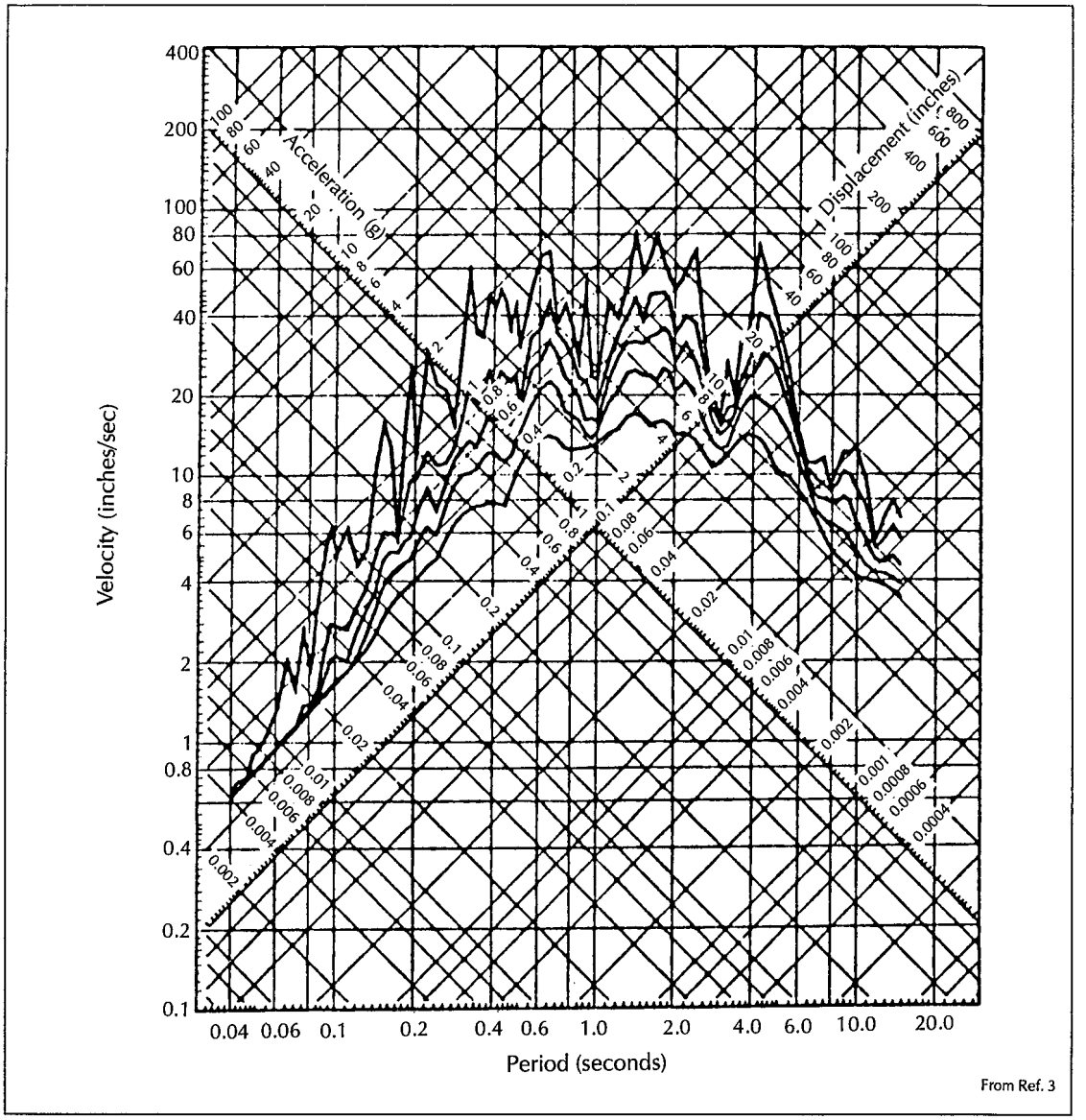
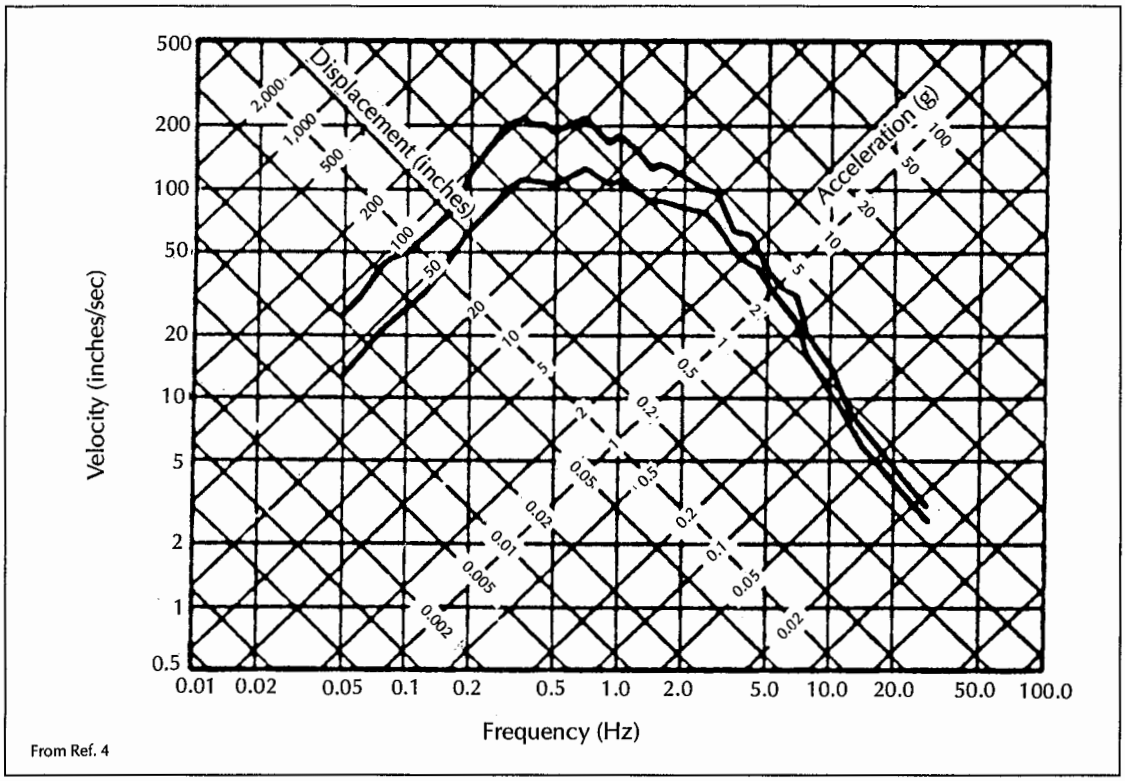


FIGURE 3. The response spectrum of north-south ground acceleration recorded at the same seven-story building during the 1971 San Fernando Earthquake (0, 0.02, 0.05, 0.10 and 0.20 of critical damping).

forced concrete frame building that was approximately five miles from the closest portion of the causative fault. The recorded building motions enabled an analysis to be made of the stresses and strains in the structure during the earthquake. Figure 2a shows the ground velocity and displacement derived from the recorded ground acceleration. Figure 2b shows the corresponding acceleration, velocity and displacement of the roof of the building located where the ground motion was recorded. Note the larger magnitudes in the diagrams of Figure 2b (the vertical scales are different). This increase in the response of the structure at the roof level over the motion of the ground itself is known as dynamic amplification. It depends very much on the vibrational characteristics of the building and the characteristic frequencies of the ground shaking at the site.

The response of a specific building to an



From Ref. 4

FIGURE 4. An averaged spectrum. The horizontal components comprise the mean and mean plus one standard deviation acceleration (2.0 percent of critical damping).

earthquake is ordinarily determined from a design response spectrum. The first step in creating a design response spectrum is determining the maximum response of a given structure to a specific ground motion (see Figure 2). The underlying theory is based entirely on the response of a single-degree-of-freedom oscillator such as a simple one-story frame. The vibrational characteristics of such a simple oscillator may be reduced to the natural frequency and the amount of damping. By recalculating the time record of response to a specific ground motion for a wide range of natural frequencies and for each of a set of common amounts of damping, the response spectra for one ground motion may be determined. It is simply the plot of the magnitude of maximum response for each combination of frequency and damping.

Using Figure 2's ground motion data, Figure 3 illustrates that the random nature of ground shaking leads to a very erratic response. It is erratic because the slightest change in period brings about a very large change in response. Different earthquake ground motions lead to response spectra with peaks and valleys at different points with respect to the natural frequency.

Computing response spectra for several different ground motions and then averaging them, based on some normalization for different amplitudes of shaking, will lead to a smoother set of spectra. Such smoothed spectra are an important step in developing a design spectrum. Figure 4 provides an example of an averaged spectrum. Note that the horizontal axes of Figures 3 and 4 differ; the former is for the period while the latter is for the cyclic frequency. Cyclic frequency is the inverse of period. Therefore, Figure 4 should be rotated about the line $f = 1$ to compare it with Figure 3. The acceleration, velocity or displacement may be obtained from Figure 3 or 4 for a structure with known frequency (period) and damping.

Multistory buildings vibrate in a more complex fashion than do simple one-degree-of-

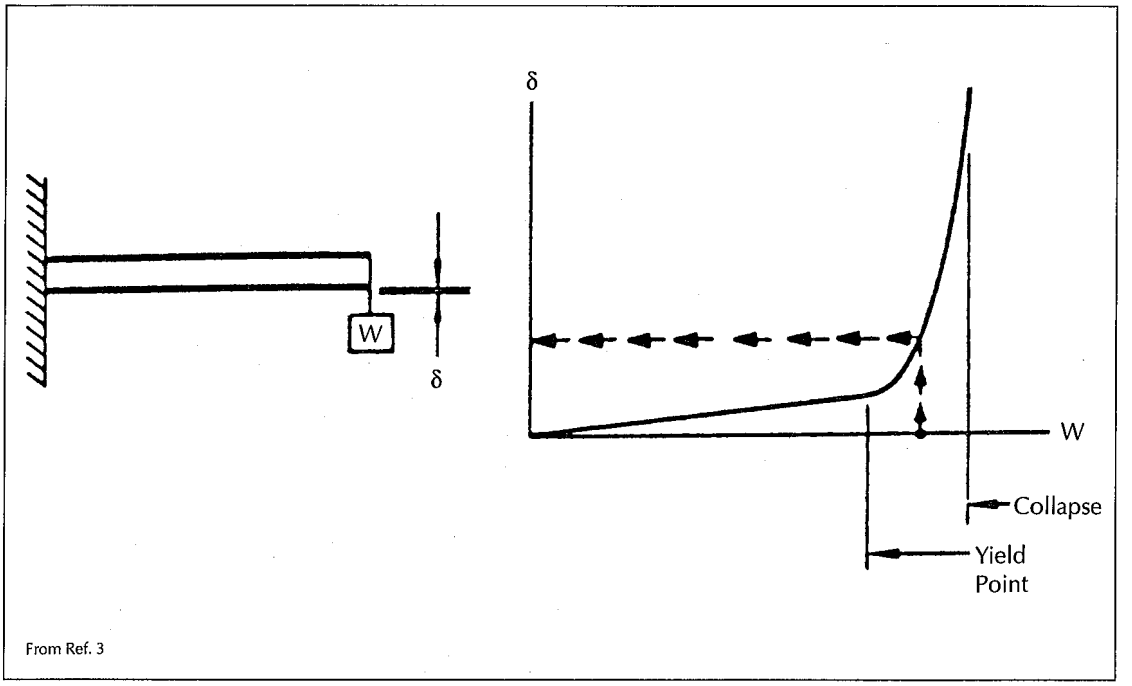


FIGURE 5. The force-controlled resistance of a steel beam.

freedom oscillators. However, the principles of modal analysis allow a reasonable approximation of the maximum response of a multi-degree-of-freedom oscillator such as a multistory building if many specific conditions are met. The procedure involves dividing the total response into a number of natural modes, modeling each mode as an equivalent single-degree-of-freedom oscillator, determining the maximum response for each mode, and then estimating the maximum total response by statistically summing the responses of the individual modes. The *Provisions* does not require consideration of all possible modes of vibration for most buildings because the contribution of the higher modes (higher frequencies) to the total response is very minor.

The soil immediately beneath a building has a significant effect on the characteristics of the ground motion and, therefore, on the building's response. This effect is accounted for in the *Provisions* by dividing the ground motions used in the averaging of response spectra into sets with similar soils. Thus, somewhat different design response spectra are specified depending on the type of soil(s) beneath the building. The *Commentary* contains a thorough explanation of this feature.

Inelastic Response. Heretofore in this discussion the elastic behavior of the building structure has been assumed. Building structures are permitted to strain beyond the elastic limit in responding to earthquake ground shaking. This principal extension beyond ordinary behavior dramatically differs from the case with design for other types of loads in which stresses and, therefore, strains are not permitted to approach the elastic limit. The reason is economic. Figure 3 shows a peak acceleration response of about 1.0 g (the acceleration of gravity) for a structure with moderately low damping — for only a moderately large earthquake. Even structures that are strong in resisting lateral forces will have a static lateral strength of only 20 to 40 percent of gravity.

The dynamic nature of earthquake ground shaking means that a large portion of the shaking energy can be absorbed by inelastic strain energy if some damage to the structure is accepted. The degree to which a member or structure may deform beyond the elastic limit is referred to as ductility. Figures 5 and 6 illustrate the large amount of strain energy that may be stored by a ductile system in a displacement-

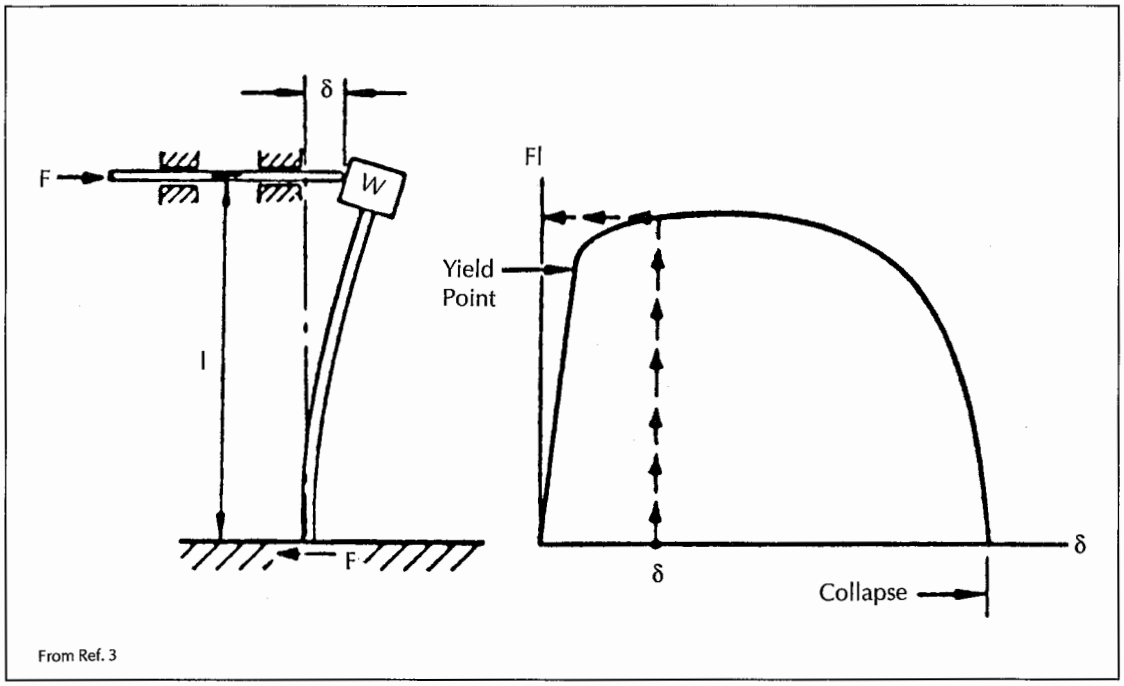


FIGURE 6. The displacement-controlled resistance of a simple oscillator with a steel column.

controlled event such as an earthquake. The two figures are plotted with the independent variables on the horizontal axis and the dependent response on the vertical axis.

Figure 5 characterizes the response to forces such as gravity weight or wind pressure. As the weight, *W*, is increased, the displacement increases until the yield point stress is reached. If *W* is given an additional increment (about 15 percent), a plastic hinge forms giving large displacements. For this kind of system, the force producing the yield point stress is close to the force producing collapse. The ductility does not produce a large increase in load capacity.

Figure 6 characterizes induced displacements such as foundation settlement or earthquake ground shaking. As the displacement is increased, the base moment, *Fl*, increases until the yield point is reached. As the displacement increases still more, the base moment increases only a small amount. The displacement can be increased ten to 20 times the yield point displacement before the system collapses under the weight, *W*. (As *W* increases, this ductility is decreased dramatically.) During an earthquake, the oscillator is excited into vibrations by the ground motion. It behaves essentially as a displacement-controlled system and can survive displacements far beyond the yield point, explaining why ductile structures can survive ground shaking that produces displacements much greater than yield point displacement.

Figures 5 and 6 should not be interpreted as a horizontal beam and a vertical column. Figure 6 would represent a beam if the load *W* was small and a column if *W* were large. The figures are used to illustrate that ductile structures have the ability to resist displacements much larger than those that first cause yield.

Different materials and different arrangements of structural members lead to different ductilities. Response spectra may be calculated for oscillators with different levels of ductility. At the risk of gross oversimplification, the following general conclusions may be drawn:

- For structures with very low natural frequencies, the acceleration response is reduced by a factor equivalent to the ductility ratio (the ratio of maximum usable displacement to effective yield displacement).

- For structures with very high natural frequencies, the acceleration response of the ductile structure is essentially the same as that of the elastic structure.
- For intermediate frequencies (which applies to nearly all buildings), the acceleration response is reduced and the displacement response is increased for the ductile structure.

Inelastic response is quite complex. Earthquake ground motions involve a significant number of reversals and repetitions of the strains. Therefore, relying on observations of the inelastic properties of a material, member or system under a monotonically increasing load until failure can be very misleading. Cycling the strain can cause the degradation of strength or stiffness, or both. Systems that have a proven capacity to maintain a stable resistance to a large number of cycles of inelastic straining are allowed to exercise a greater portion of their ultimate ductility in designing for earthquake resistance.

The building characteristics that are important for determining its seismic response are natural frequency, damping, ductility and stability of resistance under repeated reversals of inelastic strain. The natural frequency is determined by the mass and stiffness of the building. Using the *Provisions*, the designer calculates, or at least approximates, the natural period of vibration (the inverse of natural frequency). Damping, ductility and stability depend primarily on the type of building system but not the size or shape of the building. Two coefficients, R and C_d, are provided to encompass damping, ductility and stability of resistance. R reduces the acceleration response from that for an elastic oscillator with a certain level of damping. C_d amplifies the elastic displacement response for ductility. Pairs of R and C_d are specified in the *Provisions* for the most common building materials and systems.

Building Materials. Timber structures nearly always resist earthquakes very well even though wood is a brittle material as far as tension and flexure are concerned. It has some ductility in compression, and its strength increases for brief loadings such as an earthquake. Conventional timber structures (plywood or board sheathing on wood framing) possess much more ductility than the basic material primarily because of the yielding of nails and other steel connection devices and the compression of the wood against the connector. These structures also possess a much higher degree of damping than is assumed in developing the basic design spectrum. Much of this damping is caused by slip at the connections. The increased strength, connection ductility and high damping combine to give such structures a large reduction from elastic response to design level. This large reduction should not be used if the structure's strength is actually controlled by bending or tension of gross timber cross section. The large reduction in acceleration combined with the light weight of timber structures make them very efficient with regard to earthquake ground shaking when they are properly connected, as evidenced by their generally good performance in earthquakes.

Steel is the most ductile of the common building materials. The moderate to large reduction from elastic response to design response allowed for steel structures is primarily a reflection of this ductility and the stability of the resistance of steel. Members subject to buckling (*e.g.*, bracing) and connections subject to brittle fracture (*e.g.*, partial penetration welds under tension) are less ductile and the *Provisions* accounts for these features in various ways. Other defects, such as lamellar tearing at thick welds, may also affect earthquake resistance.

Reinforced concrete achieves ductility through careful limits on steel in tension and concrete in compression. Reinforced concrete beams with common proportions can possess greater ductility under monotonic loading than common steel beams, in which local buckling is usually a limiting factor. However, providing stability of the resistance to reversed inelastic strains requires uncommon detailing. The reduction factors from elastic response to design response vary widely, depending on the detailing for stable and assured resistance. The *Commentary* explains how controlling premature shear failures in members and joints, buckling of compression bars, concrete compression failures through confinement, the sequence of

plastification and other factors lead to larger reductions from the elastic response.

Masonry represents a more diverse category of building materials than wood, steel or reinforced concrete. However, less is known about its inelastic response characteristics. For certain types of members (*e.g.*, pure cantilever shear walls), reinforced masonry behaves in a fashion similar to reinforced concrete. The nature of the masonry construction, however, makes it difficult, if not impossible, to apply some of the methods (*e.g.*, confinement of compression members) used with reinforced concrete to increase ductility and stability. Further, the discrete differences between mortar and the masonry unit create additional failure phenomena. Therefore, the reduction factors for reinforced masonry are not quite as large as those for reinforced concrete. Unreinforced masonry possesses little ductility or stability, and very little reduction from the elastic response is permitted.

Building Systems. Three basic lateral load resisting elements — walls, braced frames and unbraced frames (moment-resisting frames) — are used as the basis for a classification of building types in the *Provisions*. Unbraced frames generally are permitted greater reductions from elastic response than walls and braced frames. In part, this latitude is given because frames are more redundant, have several different locations with approximately the same stress levels and because the beam-column joints often exhibit an ability to maintain a stable response through many cycles of reversed inelastic strains. Connection details often make the development of ductility difficult in braced frames, and buckling of compression members is another factor that limits their inelastic response. Eccentrically braced steel frames are a new development designed to overcome these shortcomings. Walls that are not load bearing are allowed a greater reduction than walls that are load bearing. Redundancy is one reason; another is that axial compression generally reduces the flexural ductility of concrete and masonry elements (although small amounts of axial compression usually improve the performance of materials weak in tension, such as masonry and concrete). Systems that combine different types of elements are generally allowed greater reductions from elastic response because of redundancy.

Engineering Philosophy

In the *Provisions* under "Purpose," it is stated that:

> "The design earthquake ground motions specified in these provisions are selected so that there is a low probability of their being exceeded during the normal life expectancy of the building. Buildings and their components and elements that are designed to resist these motions and that are constructed in conformance with the requirements for framing and materials contained in the following chapters may suffer damage but should have a low probability of collapse due to seismic-induced ground shaking."

The two points to be emphasized are that damage is to be expected when an earthquake (equivalent to the design earthquake) occurs and that the probability of collapse is not zero.

The basic structural criteria are strength, stability and distortion. The yield-level strength provided must be at least that required by the design spectrum (which is reduced from the elastic spectrum). The stability criterion is imposed by amplifying the effects of lateral forces for the destabilizing effect of lateral translation of the gravity weight (termed the *P-delta effect).* The distortion criterion is stated in the form of a limit on interstory drift and is calculated by amplifying the linear response to the design spectrum by the factor C_d to account for inelastic behavior.

Yield-level strengths are easily obtained for steel and concrete structures from common design standards. However, design standards for timber and masonry are based on allowable stress concepts that are not consistent with the basis of the reduced design spectrum. The *Provisions* document stipulates adjustments to common reference standards for timber and masonry to arrive at a strength level equivalent to yield and compatible with the basis of the design spectrum. Most of these adjustments are simple factors to be applied to conventional allowable stresses. Other common standards for earthquake-resistant design are based on

allowable stresses.

The *Provisions* recognizes that the risk presented by a particular building is a combination of the seismic hazard at the site and the consequence of failure, due to any cause, of the building. A classification system, based on the use and size of the building and called the Seismic Hazard Exposure Group (SHEG), quantifies that risk. A combined classification called the Seismic Performance Category (SPC) incorporates both the seismic hazard and the SHEG. The SPC is used throughout the *Provisions* for decisions regarding the application of various specific requirements.

Calculation Procedures

The *Provisions* sets forth two procedures for determining the force effect of ground shaking — an equivalent static force procedure and a dynamic modal analysis procedure. Alternate procedures such as preparing design spectra for a specific site, elastic time-history analysis or inelastic time-history analysis are permitted; however, they are subject to certain limits. Both specified methods are based on the same specified spectrum (the specification is in terms of the equation for the total lateral force or "base shear"). The entire reduction from elastic spectrum to design spectrum is accomplished by dividing the elastic spectrum by the coefficient *R*, which ranges from 1.25 to 8.0. The specified elastic spectrum is based on a damping level at five percent of critical damping. The resulting forces are compared to the full strength of the members and are not reduced by a factor of safety.

Equivalent Static Force Procedure. With this procedure, the level of the design spectrum is set by determining the appropriate values of basic seismic acceleration, the appropriate soil profile type and the value for *R*. The particular acceleration for the building is determined from this spectrum by selecting a value for the natural period of vibration. Equations that require only the height, width and type of structural system are given to approximate the natural period for various building types. Calculating a period based on an analytical model of the structure is encouraged, but limits are placed on the results of such calculations. These limits prevent the use of a very flexible model in order to obtain a large period and corresponding low acceleration. With the acceleration, the base shear is obtained from the total effective mass of the building, which is generally the total permanent load.

Once the total lateral force is determined, the procedure specifies how this force is to be distributed along the height of the building. This distribution is based on the results of dynamic studies of relatively uniform buildings, and it is intended to give an envelope of shear force at each level that is consistent with these studies. Particularly in tall buildings, this set of forces will produce an envelope of gross overturning moment that is larger than the dynamic studies indicate is necessary. Therefore, the procedure permits reducing such moments for tall buildings.

With one exception, the remainder of the equivalent static force analysis is basically a standard structural analysis. That exception accounts for uncertainties in the location of the center of mass, uncertainties in the strength and stiffness of the structural elements and rotational components in the basic ground shaking. This concept is referred to as horizontal torsion. The *Provisions* requires that the center of force be displaced from the calculated center of mass by an arbitrary amount in either direction.

Dynamic Modal Analysis Procedure. In many respects, this procedure is very similar to the equivalent static force procedure. The primary difference is that the natural period and corresponding deflected shape must be known for several of the natural modes of vibration. The period and deflected shapes are calculated from a mathematical model of the structure. The procedure requires including all modes with periods greater than 0.4 seconds and at least three modes (in the case of two-story buildings, only two modes are used). The base shear for each mode is determined from a design spectrum that is essentially the same as that for the static procedure. The distribution of forces, and the resulting story shears and overturning moments, are determined for each mode directly from the procedure. Total values for subsequent analysis and design are determined by taking the square root of the sum of the squares for each mode. This summation gives a statistical estimate of maximum response when the participation of the various

modes is random. If two or more of the modes have very similar periods, more advanced techniques for summing the values should be considered.[1] The sum of the absolute values is always conservative.

A lower limit to the base shear determined from the modal analysis procedure is specified based on the static procedure and the approximate periods specified in the static procedure. When this limit is violated, all results are scaled up in direct proportion. The consideration of horizontal torsion is the same as for the static procedure. Because the forces applied at each story, the story shears and the overturning moments are separately obtained from the summing procedure, the results are not statically compatible (*i.e.*, the moment calculated from the story forces will not match the moment from the summation). Early recognition of this will avoid considerable problems in later analysis and checking.

Comparison of the Force Effect Procedures. For buildings that are very uniform in a vertical sense, the two procedures yield very similar results. The modal analysis method is better for buildings having unequal story heights, stiffnesses or masses. The modal procedure is required for such buildings in higher hazard zones. Both methods are based on purely elastic behavior and, thus, neither will give a particularly accurate picture of behavior in an earthquake approaching the design event. The yielding of one component during an earthquake leads to the redistribution of the forces within the structural system. This redistribution may be very significant; yet, neither specified procedure accounts for it.

Both methods require considering the stability of the building as a whole. The technique is based on the elastic amplification of horizontal displacements created by the action of gravity on the displaced masses. A simple factor is calculated, and when the amplification exceeds about ten percent, it must be taken into account in designing member strengths. The technique is referred to as *P-delta* analysis and only approximates stability at inelastic response levels.

Nonstructural Elements of Buildings

Severe ground shaking often results in considerable damage to the nonstructural elements of buildings. Damage to nonstructural elements can be a hazard to life in and of itself, as in the case of heavy partitions or facades, or it can create a hazard if the nonstructural element ceases to function, as in the case of a fire suppression system. Some buildings, such as hospitals and fire stations, need to be functional immediately following an earthquake; therefore, many of their nonstructural elements must remain undamaged.

Damage to and from nonstructural elements is considered in three ways in the *Provisions*:

- Indirect protection is provided by an overall limit on structural distortion. However, the limits specified may not offer much protection to brittle elements that are rigidly bound by the structure.
- Many components are required to be anchored for an equivalent static force.
- Some elements are required to be specifically designed in order to accommodate specific structural deformations or seismic forces (the total element, not just its anchorage).

The force calculations are quite simply rendered as acceleration times mass. In most cases, the acceleration represents three factors: the seismic hazard of the location, the nature of the response of the element and the necessity for superior performance. The latter two factors are taken from a classification of elements in the *Provisions* that is more extensive than any contained in past design standards. In certain cases, added factors are introduced to account for the amplification of ground shaking in the upper stories of buildings and for the possibility of resonance between the structure as a whole and the equipment being supported.

Quality Assurance

Since strong ground shaking tends to reveal hidden flaws or "weak links" in buildings, the *Provisions* contains detailed requirements for quality assurance during construction. Loads experienced during construction provide a significant test of the likely performance of ordinary buildings under gravity loads. Tragically,

mistakes occasionally will pass this test only to cause failure later, but their occurrence is fairly rare. No comparable proof test exists for horizontal loads, and experience has shown that flaws in construction show up as distress and failure due to an earthquake in a disproportionately large share of buildings. Damage also occurs because the design is based on excursions into inelastic straining, which is not the case for response to other loads.

Quality assurance provisions require a systematic approach that emphasizes documentation and communication. The designer who conceives the systems to resist the effects of earthquake forces must identify the elements that are critical for successful performance and must specify the testing and inspection necessary to ensure that those elements are actually built to perform as intended. Minimum levels of testing and inspection are specified in the *Provisions* for various types of systems and components.

The *Provisions* also requires that the contractor and building official be aware of the requirements specified by the designer. Furthermore, the necessary inspection and testing must be performed by technically qualified personnel who can communicate the results of their work to all concerned parties. In the final analysis, there is no substitute for a sound design, soundly executed.

Historical Development of U.S. Seismic Codes

Much of the history of the development of seismic code provisions in the United States is the direct response to damaging earthquakes. The bulk of this discussion focuses on developments within this country; however, a few significant early events outside the U.S. are included for perspective. Also, many of the developments in later years are captioned by the time of their actual occurrence, rather than being lumped together with the preceeding earthquake that stimulated the development.

1755 Lisbon Earthquake: Ground-Shaking Waves. This earthquake was a very significant one. It actually occurred in the Atlantic Ocean near the Azores Islands. It caused great damage in Lisbon through ground shaking, tsunami and subsequent fire. The earthquake was felt throughout most of western Europe, including Scandinavia. A significant seismic observation made at the time was that the ground moved in waves and that the nature of these waves changed with time — they were closer together early in the disturbance and further apart later in the disturbance.

1906 San Francisco Earthquake: Lateral Force From Wind. This earthquake was extremely destructive. Building regulations were in place in San Francisco and other major American cities at the time. However, much of the public discussion of the earthquake centered on the fire that followed. The building regulations that had been developed over the preceding century were mostly aimed at preventing conflagration in cities. The only structural safety requirement that appeared in San Francisco building code as the result of the 1906 earthquake was a provision that all buildings were to be designed to resist a horizontal force equivalent to 30 pounds per square foot of wind on the face of a building.

1911 Messina, Italy, Earthquake: Equivalent Static Inertial Force. Building regulations were enacted following this earthquake, near the straits that separate Italy from Sicily. These regulations stipulated that a building be designed for a static horizontal force equal to ten percent of its weight. These regulations represent the first instance that the fundamental law of dynamics (force equals mass times acceleration) was recognized in the design of a building in order to resist the dynamic effect of ground shaking.

1923 Tokyo (Kanto) Earthquake: Prediction. This earthquake is the first documented instance of successfully predicting a large earthquake. Prior to its occurrence, scientists in Japan had noted that a gap existed in the areas influenced by historic earthquakes in Japan. The seismic gap prediction is one of the most well founded predictive methods in use today, particularly in areas subject to strong subduction zone earthquakes. (Undoubtedly, there were significant building regulations that must have developed in Japan following this earthquake, particularly given discussion by Japanese engineers and scientists of proposed U.S. earthquake regulations in the early 1950s.

However, that information has not been reviewed directly for this discussion.)

1925 Santa Barbara: Scientific Study of Ground Motions. This earthquake stimulated a scientific approach to providing earthquake-resistant buildings. Congress instructed the U.S. Coast and Geodetic Survey (USCGS) to study strong motion seismology. The USCGS developed strong motion seismographs and placed them in many likely locations.

1927 Uniform Building Code: Inertial Force, Soil Effects in the U.S. The first edition of the Uniform Building Code (UBC) was published in 1927 by the forerunner of the International Conference of Building Officials (ICBO). The code was quickly adopted by many cities along the west coast. Its appendix contained provisions for the structural design of a building to resist an earthquake. The basic provision was that buildings on firm ground must be designed to resist a horizontal force equal to 7.5 percent of a total dead *and* live load of the building. Firm ground was defined as soil for which the allowable foundation bearing pressure was two tons per square foot or more. For soils with a lower bearing capacity and for all buildings founded on piles, the horizontal force was to be ten percent of the total dead and live load of the building. The design stresses were permitted to be increased when considering earthquake loads by one-third to one-half the normal values, depending on the component and material.

1933 Long Beach Earthquake: Reinforced Masonry & Inspection. The first instrumental records of strong ground shaking were obtained. However, they were flawed in that the amplitude exceeded the range of the instruments that the USCGS had constructed and installed. Many changes occurred in building codes. The city of Los Angeles required design for eight percent of the sum of the dead load plus half of the live load. The UBC soon adopted this regulation for good soils, and sixteen percent for poor soils. Masonry was required to have a minimum amount of reinforcement, due to the poor performance of unreinforced masonry. The earthquake shocked the officials in California, particularly because so many school buildings were destroyed. If the earthquake occurred an hour earlier, it would have caused a large loss of life. As a direct result, the state of California passed the Field and Riley Acts. The Field Act covered the design and construction of schools, requiring slightly higher forces for low-rise masonry buildings than the UBC, but perhaps most importantly instituting a rigid system of design review and quality assurance inspection during construction. In later years, this act has been expanded to include hospitals and other public buildings. The Riley Act applied to most other types of buildings and required a minimum horizontal force for design of two percent of the weight of the building. From this point on, construction in those cities in California that had not previously adopted the earthquake regulations from the UBC was required to provide for earthquake forces.

1940 El Centro, California, Earthquake: Strong Ground Motion Record. For the first time, a relatively close record of strong ground shaking from a large earthquake (magnitude 7.1) was obtained. The USCGS had redesigned their instruments following the Long Beach earthquake. As the technology for analyzing the response of a structure developed over the ensuing three decades, this singular record became the most studied and analyzed earthquake ground motion record in existence. Within ten years, the response of a single-degree-of-freedom elastic oscillator (representative of a one-story building) had been computed for this earthquake record by several leading academicians. These computations showed accelerations of five to fifty times the magnitude of the current design forces. It was nearly ten more years before the substantial differences between the elastic computations and building design practice were rationally discussed in the literature, and another fifteen years before building regulations were written that explicitly dealt with the substantial differences.

1943 City of Los Angeles Building Code: Dynamic Properties. For the first time, the seismic acceleration of the building was related to the dynamic properties of the building in addition to mass. The relation, which was approximate, set the acceleration to be inversely proportional to the number of stories of the building. Since the provision was applied one story at a time, it resulted in an unequal distribution of the total force with respect to the height, giving higher

accelerations for the upper stories than the lower stories. This basic provision soon found its way into the UBC. It was still in use in other model building codes until 1975.

1952 San Francisco Joint Committee: Modal Analysis, No Soil Effect, Vertical Distribution, Overturning, Framing System. A committee jointly appointed by the San Francisco section of the American Society of Civil Engineers (ASCE) and the Structural Engineers Association of Northern California prepared a suggested seismic code for the city of San Francisco. It was eventually adopted by the city with some increases in coefficients a few years later. The committee's justification for the provisions and the discussion stimulated by its publication are very informative.[5, 6] The use of modal analysis to approximate the dynamic response of a multistory building was discussed, as well as a response spectrum from a 1940 El Centro Earthquake record. The design acceleration was directly related to the shape of this response spectrum, but the magnitude of acceleration was not. The justification for the provisions did not adequately access the reasons for the difference between the inertial forces computed for the El Centro ground motion and design values that had given good performance. The discussion by Martel, Housner and Alford attributed at least a part of the difference to high levels of damping in buildings.[6] No adjustment to design accelerations were made for different types of soils, even though the issue was raised by several of the discussers.

The distribution of seismic accelerations along the height of the building recognized the dynamic response of the real structure; the triangular distribution recommended fairly represents the behavior of a structure vibrating in its first mode. It is interesting to note that the earlier city of Los Angeles formula did a better job of accounting for the effects of higher modes at the tops of buildings than the strictly triangular distribution. The triangular distribution, with modifications for taller buildings, is still used today. The joint committee also required resistance to the static overturning moment produced by the horizontal seismic forces, except that no increase in moment need be taken more than ten stories below the top of the building, in recognition of the effects of higher modes on taller structures.

Los Angeles imposed a height limit of 13 stories on buildings, but San Francisco did not. The joint committee therefore imposed additional requirements on tall buildings requiring the construction of a moment-resisting structural framework capable of resisting at least a portion of a seismic force within tall buildings. Their provisions also allowed the walls and bracing systems of buildings to be designed for a lower force if a moment-resisting framework is in place. The provisions appear to be the first to require that the designer take explicit account of horizontal torsion resulting from an eccentricity between the lateral force resisting system and the center of mass.

1956 World Conference on Earthquake Engineering: Research. This conference was the first in a continuing series and highlighted the progress of serious study in developing an understanding of earthquakes and structural performance.

1957 Mexico City Earthquake: Design Using Dynamic Analysis. The dramatic success of the just constructed 43-story Latino-Americano Tower designed according to dynamic principles highlighted a trend of academic research results moving into practice.

1960 Structural Engineers Association of California Requirements: Structural System Performance. Engineering associations in southern and northern California formed a joint committee in 1957 to develop uniform seismic code provisions. The first edition of the Structural Engineers Association of California (SEAOC) regulations was quickly incorporated into the UBC, which also added the aspect of zoning by inserting a factor that gave smaller design forces for areas with records of lower seismicity than California. The design acceleration was explicitly related to the period of the building, and the mass was taken as dead load only, except for storage and warehouse occupancies. The resulting spectrum of design accelerations was somewhat more similar to the 1943 Los Angeles code than the 1952 joint committee report.

The most substantial change was the introduction of a factor to represent the performance of different building systems. Systems that had

performed well, such as complete moment-resisting space frames with substantial redundancy and ductility, were permitted to be designed for smaller forces. Systems that had experienced substantial difficulty, such as bearing wall buildings, were required to be designed for somewhat higher forces. All masonry and concrete were required to have at least a minimum amount of reinforcement.

The building was required to be designed for the static resultant overturning moment from the horizontal forces. However, a reduction factor was incorporated to recognize the effect of higher modes. This factor permitted the design moment to be reduced to as little as one-third of the static resultant moment. The distribution of force with respect to height of the building was essentially the same as the triangular distribution used by the 1952 San Francisco committee, with the exception that a portion of the force was applied independently at the top of the structure for medium and high rise buildings, to represent the effect of higher modes.

1961 "Design of Multi-Story Reinforced Concrete Building for Earthquake Motions": Ductile Concrete. This text book, written for the practicing engineer, introduced the concepts of inelastic response to earthquake motions in clear detail for the first time. It was also one of the first clear expositions of dynamic modal analysis for use by practicing engineers. It emphasized the concepts by which ductility can be provided with reinforced concrete columns and beams.

1964 Alaska Earthquake: No Instrumental Data. This earthquake was an extremely large and destructive one. Unfortunately, no instrumental record of the strong ground motion was obtained. Building code concepts for torsional response of eccentric buildings, anchorage of cladding to buildings and details for tieing the structure together were reinforced by real performance data. Substantial landslides and ground movement occurred.

1964 Niigata, Japan, Earthquake: Liquefaction. In this earthquake, extensive liquefaction created dramatic failures of reinforced concrete apartment blocks that sank and tilted into sandy deposits that became quicksand.

1966 SEAOC Requirements: Ductile Concrete, Detailing. The concrete ductile frame was introduced into the building code as a system that was permitted for tall buildings on an equivalent basis with steel frames. The requirements for detailing columns and beams in the frame to provide ductility were also included, beginning the tradition of paying extensive attention to detailing to deliver assumed ductility. Design forces for tanks and other non-building structures were increased over that of previous codes.

1967 Caracas, Venezuela, Earthquake: Nonstructural Infill, Overturning. The collapse of several high (about 20 stories) concrete frame buildings illustrated the hazards of rigid nonstructural walls in the upper stories of a building with open framework at the lowest story. The stiff walls modified the assumed dynamic response and focused the inelastic response in the first story, which collapsed. The failures also demonstrated flaws in the previous theory concerning reduction in the overturning moment because of the apparent failure of corner columns and buildings due to high compression. The reduction for overturning moment was removed from the SEAOC requirements in 1969.

1971 San Fernando Earthquake: Extensive Test of Design and Construction Practices, Federal Response. Extensive performance data were obtained on buildings designed according to seismic codes over the prior 35 years. Although the large loss of life occurred in buildings that predated seismic codes, the performance of more modern buildings was unsatisfactory in many respects. Quantified ground-shaking and dynamic building response data for further study increased by an order of magnitude. The federal government developed a keen and long lasting interest in earthquake hazard mitigation.

1972 Workshop: Upgrade Design Standards. A workshop was held in response to the San Fernando Earthquake. One of the recommendations made by the workshop was that the state of practice for building design should be brought up to the state of knowledge existing in the research community.

1974 SEAOC Requirements: Increased Forces, Occupancy Importance, Soil Factor. As a first response in the codes to the San Fernando Earth-

quake, design accelerations were raised for all buildings and an importance factor was introduced to provide even higher levels of safety for hospitals and other specially important buildings. A site factor was introduced to raise the design acceleration for those buildings with periods of vibration near the computed period of vibration of the soil profile at the site. The maximum increase was 50 percent and it was based on the concept of resonance. It exerted the most change for large buildings on soft to medium soils. A quantitative limit on lateral drift of the building in response to the seismic forces was introduced for the first time. The changes were all incorporated into the 1976 UBC.

1974 Applied Technology Council Report ATC 2: Dual Spectra Method. A two-level response spectrum technique for design, based on concepts used within the nuclear power industry, was applied to building design. It was concluded that a single design spectrum would be adequate for building code use.

1976 (published 1978) Applied Technology Council Report ATC 3-06: Probabilistic Ground Accelerations, Realistic Response Accelerations, Inelastic Factors, Strength Design, Ground Motion Attenuation. This report by the Applied Technology Council (ATC) entitled, "Tentative Provisions for the Development of Seismic Regulations for Buildings," was the first major step in the process envisioned in the 1972 workshop. A completely new set of building design provisions was developed based on the principles of dynamic response with explicit recognition of inelastic action. The ground motion maps showed realistic accelerations derived on a probabilistic basis. The elastic dynamic response was then computed either by an equivalent static method similar to traditional methods or by modal analysis and then reduced for the ductility and damping of the particular building system. Structures were to be proportioned by strength methods, and extensive requirements for detailing were included. The maps included the effect of less rapid attenuation of long period waves to protect tall buildings from distant earthquakes. The provisions were intended to be applicable nationwide, not simply in California, with a downward scaling elsewhere.

1977 Earthquake Hazards Reduction Act: Federal Support & Direction. Congress passed the law that led to the establishment of the NEHRP. More money became available for research, and more federal control was exerted. Building codes were specified as a target for improvement.

1979 Building Seismic Safety Council: Response to ATC 3-06. The BSSC was formed in order to provide a forum to debate a discussion of the issues presented by the ATC 3-06 publication. Committees were assembled and, in 1980, a review was completed that called for significant fine tuning. From 1982 to 1984, a series of trial designs were conducted to assess the economic and social impacts of the new seismic provisions, after which further modifications were made.

1985 NEHRP Recommended Provisions: ATC 3-06 Passes a Major Hurdle. The BSSC officially adopted the revised ATC 3-06 provisions as *recommended* rather than *tentative*. Updated editions were issued in 1988 and 1991. Significant changes included a finer classification for detailing requirements, design provisions for braced steel frames (both eccentric and concentric) and more guidance for irregular buildings.

1985 Mexico City Earthquake: Extreme Site Effect. This strong distant earthquake dramatically illustrated the effect of soils and local geology. By 1988 a new category for very soft soils was introduced into BSSC and SEAOC provisions.

1988 New SEAOC Requirements Incorporated Into the UBC. SEAOC officially adopted new seismic provisions in 1987, which ended up in the 1988 UBC. The provisions were essentially the same as the 1988 *NEHRP Recommended Provisions.* The major difference was that design force levels were scaled down by about one-third so existing allowable stress procedures for member design could be used. Also, the difference between close and distant earthquake ground shaking was not included.

1988 Armenia Earthquake: Importance of Detailing, Site Effect. Design forces in the Soviet Union were derived quite similarly to those in the U.S. However, the Soviet design standards did not include much detail to ensure the assumed ductility. An extreme disparity in the performance of two precast building systems highlighted the

folly. Also, the effect of soft soil underlying a crust of weak volcanic rock surprised local officials.

1989 Loma Prieta (World Series) Earthquake: Performance Test. Tremendous quantities of data were gathered and have not yet been fully analyzed. However, the basic approach of building design taken since the San Fernando Earthquake was affirmed.

1991 NEHRP Provisions Into Model Codes: Upgrade Practices in the East, Midwest and South. Official moves were taken to incorporate up-to-date seismic provisions in the Building Officials and Code Administrators' (BOCA) National Building Code and the Standard Building Code, as well as voluntary national consensus standards.

The System of Building Codes in the U.S.

In the United States, the authority to regulate the construction and use of buildings rests with the state governments because the federal Constitution does not specifically assign that authority to the federal government. Traditionally, however, the states have not exercised that authority, leaving its administration to local governments. Even though many states have enacted statewide building codes in recent years, most legal building codes are the laws of cities and towns. Each one of these codes has the potential to be, and often is, unique. While there are unifying influences on building codes that tend to reduce the diversity, there are a great many diverse building codes.

Because the drafting of a building code is usually far too big a job for any but the largest cities to undertake, model codes are in widespread use as the basis for most communities. Individual jurisdictions often adopt one of the models as their legal code, but frequently the models are amended, so diversity still exists. There are three model building codes produced in the U.S.:

- *The Uniform Building Code,* published by the International Conference of Building Officials.
- *The Standard Building Code,* published by the Southern Building Code Congress International.
- *The BOCA National Building Code,* published by the Building Officials and Code Administrators International.

Produced by associations of building officials who have joined together in an effort to produce better codes with less duplication of effort, these model codes are influenced by essentially the same sectors of society as the local codes, with some obvious differences in scale. For example, an individual engineer might try to influence a decision on a local building code, whereas a national association of engineers would exert an analogous influence on a model code. Although the three model code associations permit nearly any interested party to participate in the committee meetings they conduct, the final decision on what the model code contains is made by vote of the association members.

There are many other building code related organizations. Some function to improve building codes, some increase the uniformity of building codes and some raise the professional stature and competence of building officials. Among these organizations are the Council of American Building Officials, the Model Code Standardization Council, the National Academy of Code Administration and the National Conference of States on Building Codes and Standards. While the current trend is moving away from the great diversity of thousands of different building codes, there is still a great distance to go.

The model code associations are not the initial authors of everything in their codes. Indeed, the bulk of the provisions governing buildings today come directly or indirectly from hundreds of national standards that deal with engineering practice, material specifications and test methods. Some standards are used as resources for the writers of model codes. For example, many of the provisions for emergency exiting in the building codes come from *The Life Safety Code,* published by the National Fire Protection Association (NFPA). (Although this document is named a code, it is not law since the NFPA is not a governmental entity. It is more properly called a standard.) Many standards are referenced in the model codes.

These standards are produced by over a thousand different committees in hundreds of different organizations. Large standards generating organizations in addition to the NFPA include: the U.S. Department of Commerce; the Institute of Electrical and Electronics Engineers; the American Society of Heating, Refrigeration, and Air Conditioning Engineers; and other professional societies. In addition, many trade associations produce standards, such as the Brick Institute of America, the American Iron and Steel Institute, and so on. The American National Standards Institute (ANSI) has generated a large number of standards for many years, but is now moving into the role of coordinating the efforts and policies of many organizations involved in generating standards.

The standards-generating committee members come from many different sectors of society. Standards can be divided roughly into two groups based on who the decision makers are and how decisions are made. The first group is characterized by:

- *Balanced* representation from all interested parties. In the case of building standards, members include materials suppliers, product producers, contractors, labor, designers, researchers, consumers and representatives of government from federal, state and local levels.
- *Consensus* as a basis for decision making, which implies substantially more than a majority, but not necessarily unanimity.
- *Due process* in the hearing and resolution of all issues, including allowing adequate time and notice for voting and the public resolution of all dissenting ballots.

ANSI standards fall into this first group, and ANSI also certifies other organizations that generate standards following procedures with these characteristics, such as the American Society of Testing and Materials, the NFPA and others.

The second group of standards are all those that are not part of the first group. The prototypical example of this group are proprietary standards issued by a trade association, as for example the *Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings*, which is published by the American Institute of Steel Construction.

The characteristics that these standards typically do not possess is balance or due process. Both kinds of standards are widely used and quite important. These model codes and standards are important cogs in the communication of information in the construction industry. They are applied less rigorously than legal building codes, or even contracts, because no force of law stands behind them. They have a much broader range of applicability than any legal code, however, because legal building codes in the U.S. do not apply over more than one state, and most are local (with some exceptions like the Federal Mobile Home Construction and Safety Standards).

Seismic Codes

The current status of earthquake provisions in U.S. codes is in a state of flux, with a pronounced east-west division. The UBC predominates in the West, and its seismic provisions are developed by SEAOC. In 1988, the UBC updated provisions based on the ATC 3-06 report. Both the *National* and *Standard* codes have historically made reference to, or incorporated, the seismic provisions of ANSI A58.1, which are essentially an abbreviated version of the SEAOC/UBC provisions. The current editions are based on the old UBC provisions. ASCE has assumed responsibility for producing what was ANSI A58.1. ASCE and the *National* and *Standard* codes are in the process of adopting BSSC's *Provisions* into their documents. Once that is complete, all the model codes and the most referenced national standards will be up to date, as recommended immediately following the San Fernando Earthquake. The most significant difference is that the SEAOC/UBC provisions are based on working stress design methods.

Most of the current model codes specify a design spectrum based on a nationwide map of a single seismic ground-shaking parameter that represents peak ground acceleration. ATC 3-06 and the *Provisions* (but not the 1988 UBC) use two maps, which gives rise to different spectral shapes. All the model codes adjust the design spectrum for soil conditions, usually requiring the engineer to select one of three or four characteristic soil profiles. Inelastic re-

sponse is accounted for by modifying the entire spectrum uniformly with a factor that depends on the structural system.

In the future, response spectral values will be mapped directly, rather than ground motion values. Also, the spectral values may be mapped for more than two periods, giving more discrimination in the shape of design spectra. It is also possible that soil factors will show more variation. The inelastic response modification factor may also show variation with period and with expected duration of strong shaking.

Equivalent static analysis is the basis for current codes, although BSSC's provisions and the 1988 UBC require planar dynamic modal analysis for certain classes of irregular structures. In the future, three-dimensional dynamic analysis will be required for additional classes of irregular structures. Time history response analysis is unlikely to be required for buildings until the art of specifying ground motion time histories has improved and the cost of computing reduces some more. Improved methods for determining inelastic demand, such as sequential yield analysis and inelastic time history analysis will find their way into codes if their viability can be proven.

Current design is based on the earthquake with a probability of occurrence of 0.2 percent in one year, producing an ultimate limit state in a member after applying the inelastic response modification factor. There may be considerable tinkering with this assumption. One likely approach that suits lower hazard zones is a dual criteria in which the elastic response to a more frequent earthquake (say, two percent per year) satisfies ultimate limit states and the inelastically reduced response to a more rare event (say, 0.05 percent per year) satisfies a collapse limit state. Another possible approach is a two-level analysis: elastic for the relatively frequent event and inelastic demand for the rare event.

Detailing requirements for toughness will necessitate some changes in the lower hazard zones. Due to the apparent large difference in demand between the relatively frequent event and the rare event, heavy structures are likely to show a higher ductility demand than is currently required. There will also be a continuation of the trend towards specifying empirical requirements based on the observed performance in real earthquakes. The new requirements related to building configuration are the best current example. Detailing rules for "new" structural systems will most probably continue to appear. The eccentrically braced steel frame is the best recent example. Coupled concrete shear walls and precast concrete systems are ripe for this development.

The seismic provisions of the Massachusetts Building Code are based on UBC provisions that predate the 1971 San Fernando Earthquake. The intensity of seismic loading required for the Massachusetts area (Seismic Zone 2) in the more recent editions of the UBC exceeds that given in the Massachusetts code, except for very tall buildings. This situation remains doubly true for the 1976 through 1985 editions of the UBC, in which the Boston area was placed in Zone 3. To make comparisons between the Massachusetts code and BSSC's *Provisions*, an adjustment for the difference between allowable stress design and strength design must be made. Once this adjustment is done, and using the original maps for the *Provisions*, the seismic loading intensity is comparable to the Massachusetts code. Using the most up to date maps, the intensity is comparable to the modern UBC.

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JAMES ROBERT HARRIS *is President of J.R. Harris & Co. in Denver, Colorado. He has a strong background in structural engineering practice and research. His experience includes the*

design or evaluation of hundreds of structures, ranging from dwellings to high-rise buildings and industrial facilities. He has worked with designs of excavation bracing, pile and pier foundations, and renovations of historic facilities. Many of his designs are sited in active seismic regions. His research is focused on the loading and response of structures, particularly for earthquake and snow loads.

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