CHAPTER 24

VIBRATION OF STRUCTURES INDUCED BY GROUND MOTION

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INTRODUCTION

This chapter discusses typical sources of ground motion that affect buildings, the effects of ground motion on simple structures, response spectra, design response spectra (also called *design spectra*), and design response spectra for inelastic systems. The importance of these topics is reflected in the fact that such characterizations normally form the loading input for many aspects of shock-related design, including seismic design. Selected material are presented which are pertinent to the design of resisting systems, for example, buildings designed to meet code requirements related to earthquakes.

GROUND MOTION

SOURCE OF GROUND MOTION

Ground motion may arise from any number of sources such as earthquake excitation[1,2] (described in detail in this chapter), high explosive,[3] or nuclear device detonations.[4] In such cases, the source excitation can lead to major vibration of the primary structure or facility and its many parts, as well as to transient and permanent translation and rotation of the ground on which the facility is constructed. Detonations may result in drag and side-on overpressures, ballistic ejecta, and thermal and radiation effects.

Other sources of ground excitation, although usually not as strong, can be equally troublesome. For example, the location of a precision machine shop near a railroad or highway, or of delicate laboratory apparatus in a plant area containing heavy drop forging machinery or unbalanced rotating machinery are typical of situations in which ground-transmitted vibrations may pose serious problems.

Another different class of vibrational problems arises from excitation of the primary structure by other sources, e.g., wind blowing on a bridge, earthquake excitation of a building, or people walking or dancing on a floor in a building. Vibration of the primary structure in turn can affect secondary elements such as mounted equipment and people located on a floor (in the case of buildings) and vehicles or equipment (in the case of bridges). A brief summary of such people-structure interaction is given in Ref. 5.

The variables involved in problems of this type are exceedingly numerous and, with the exception of earthquakes, few specific well-defined measurements are generally available to serve as a guide in estimating the ground motions that might be used as computational guidelines in particular cases. A number of acceleration-vs.-time curves for typical ground motions arising from the operation of machines and vehicles are shown in Fig. 24.1. Another record arising from a rock quarry blast is shown in Fig. 24.2. Although the records differ somewhat in their characteristics, all can be compared directly with similar measurements of earthquakes, and response computations generally are handled in the same manner.

In most cases, to analyze and evaluate such information one needs to (1) develop an understanding of the source and nature of the vibration, (2) ascertain the physical characteristics of the structure or element, (3) develop an approach for modeling and analysis, (4) carry out the analysis, (5) study the response (with parameter variations if needed), (6) evaluate the behavior of service and function limit states, and (7) develop, in light of the results of the analysis, possible courses of corrective action, if required. Merely changing the mass, stiffness, or damping of the structural system may or may not lead to acceptable corrective action in the sense of a reduction in deflections or stresses; careful investigation of the various alternatives is required to change the response to an acceptable limit. Advice on these matters is contained in Refs. 3, 6, and 7.

RESPONSE OF SIMPLE STRUCTURES TO GROUND MOTIONS

Four structures of varying size and complexity are shown in Fig. 24.3: (*A*) a simple, relatively compact machine anchored to a foundation, (*B*) a 15-story building, (*C*) a 40-story building, and (*D*) an elevated water tank. The dynamic response of each of the structures shown in Fig. 24.3 can be approximated by representing each as a simple mechanical oscillator consisting of a single mass supported by a spring and a damper as shown in Fig. 24.4. The relationship between the undamped angular frequency of vibration $\omega_n = 2\pi f_n$, the natural frequency f_n, and the period T is defined in terms of the spring constant k and the mass m:

$$\omega_n^2 = \frac{k}{m} \tag{24.1}$$

$$f_n = \frac{1}{T} = \frac{\omega_n}{2\pi} = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \tag{24.2}$$

In general, the effect of the damper is to produce damping of free vibrations or to reduce the amplitude of forced vibrations. The damping force is assumed to be equal to a damping coefficient c times the velocity $\dot{u}$ of the mass relative to the ground. The value of c at which the motion loses its vibratory character in free vibration is called the *critical damping coefficient;* for example, $c_c = 2m\omega_n$. The amount of damping is most conveniently considered in terms of the fraction of critical damping, ζ [see Eq. (2.12)],

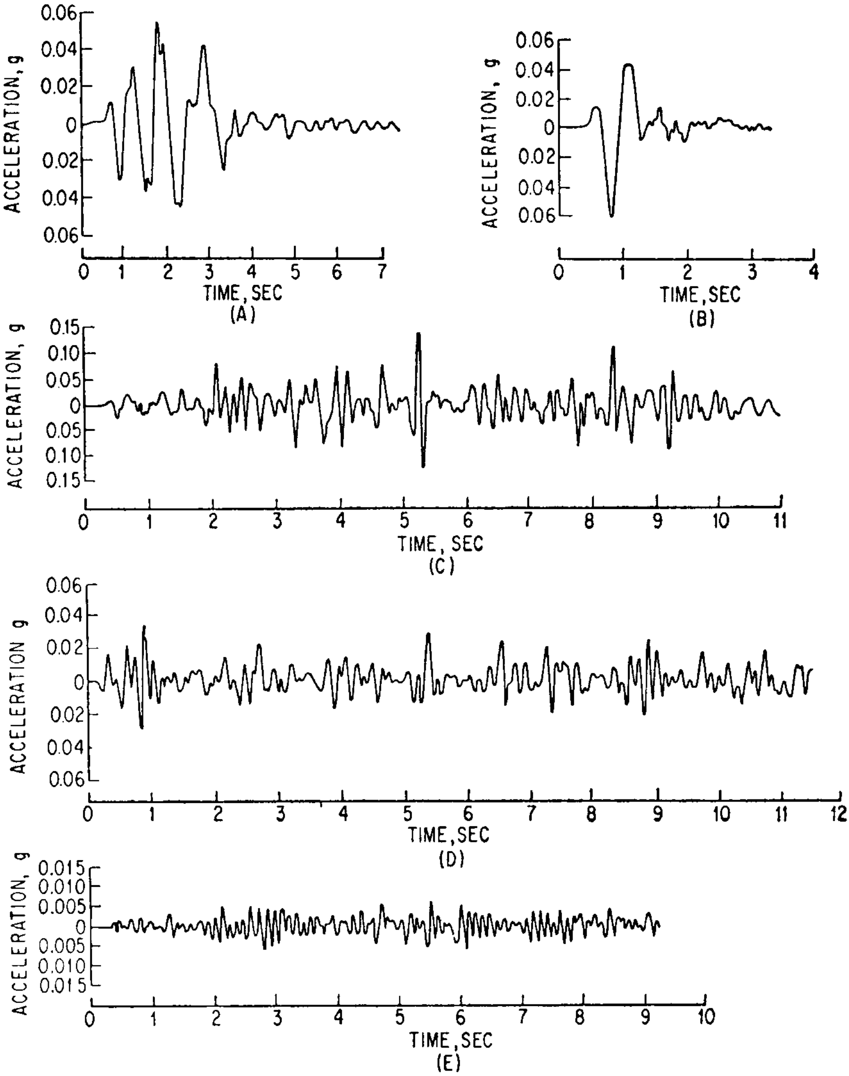

FIGURE 24.1 Ground-acceleration-vs.-time curves for typical machine and vehicle excitations. (*A*) Vertical acceleration measured on a concrete floor on sandy loam soil at a point 6 ft from the base of a drop hammer. (*B*) Horizontal acceleration 50 ft from drop hammer. The weight of the drop hammerhead was approximately 15,000 lb, and the hammer was mounted on three layers of 12- by 12-in. oak timbers on a large concrete base. (*C*) Vertical acceleration 6 ft from a railroad track on the well-maintained right-of-way of a major railroad during passing of luxury-type passenger cars at a speed of approximately 20 mph. The accelerometer was bolted to a 2- by 2-in. by 2½-in. steel block which was firmly anchored to the ground. (*D*) Horizontal acceleration of the ground at 46 ft from the above railroad track, with a triple diesel-electric power unit passing at a speed of approximately 20 mph. (*E*) Horizontal acceleration of the ground 6 ft from the edge of a relatively smooth highway, with a large tractor and trailer unit passing on the outside lane at approximately 35 mph with a full load of gravel.[6]

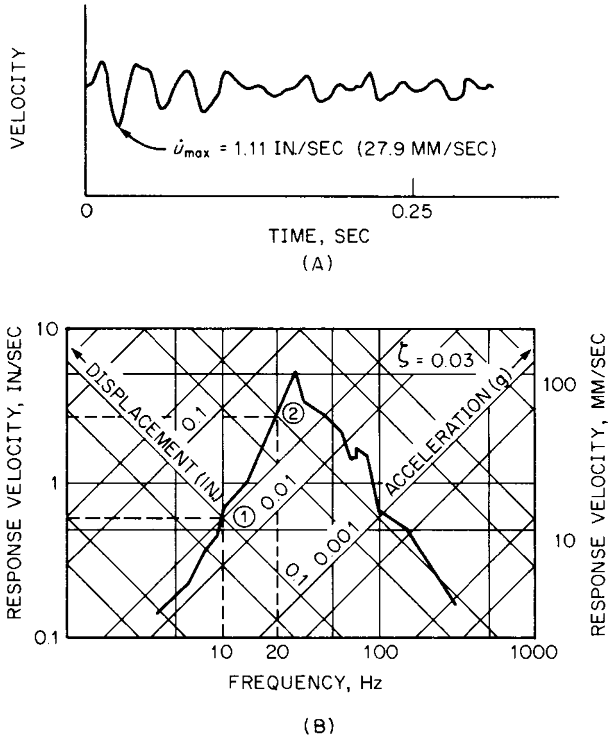

FIGURE 24.2 Typical quarry blast data. (*A*) Time-history of velocity taken by a velocity transducer and recorder. (*B*) Corresponding response spectrum computed from the record in (*A*) using Duhamel's integral.[3]

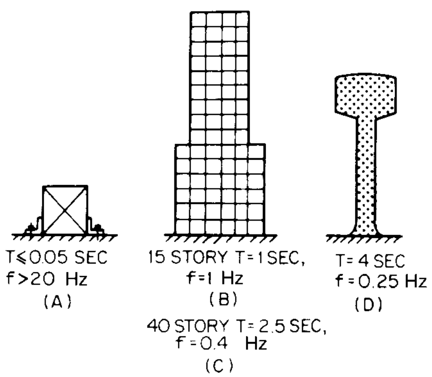

FIGURE 24.3 Structures subjected to earthquake ground motion. (*A*) A machine anchored to a foundation. (*B*) A 15-story building. (*C*) A 40-story building. (*D*) An elevated water tank.

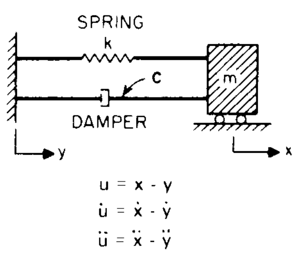

FIGURE 24.4 System definition; the dynamic response of each of the structures shown in Fig. 24.3 can be approximated by this simple mechanical oscillator.

$$\zeta = \frac{c}{c_c} = \frac{c}{2m\omega_n} \tag{24.3}$$

For most practical structures ζ is relatively small, in the range of 0.005 to 0.2 (i.e., 0.5 to 20 percent), and does not appreciably affect the natural period or frequency of vibration (see Refs. 1*b* and 8).

EARTHQUAKE GROUND MOTION

Strong-motion earthquake acceleration records with respect to time have been obtained for a number of earthquakes. Ground motions from other sources of disturbance, such as quarry blasting and nuclear blasting, also are available and show many of the same characteristics. As an example of the application of such time-history records, the recorded accelerogram for the El Centro, California, earthquake of May 18, 1940, in the north-south component of horizontal motion is shown in Fig. 24.5. On the same figure are shown the integration of the ground acceleration a to give the variation of ground velocity v with time and the integration of velocity to give the variation of ground displacement d with time. These integrations normally require baseline corrections of various sorts, and the magnitude of the maximum displacement may vary depending on how the corrections are made. The maximum velocity is relatively insensitive to the corrections, however. For this earthquake, with the integrations shown in Fig. 24.5, the maximum ground acceleration is 0.32g, the maximum ground velocity is 13.7 in./sec (35 cm/sec), and the maximum ground

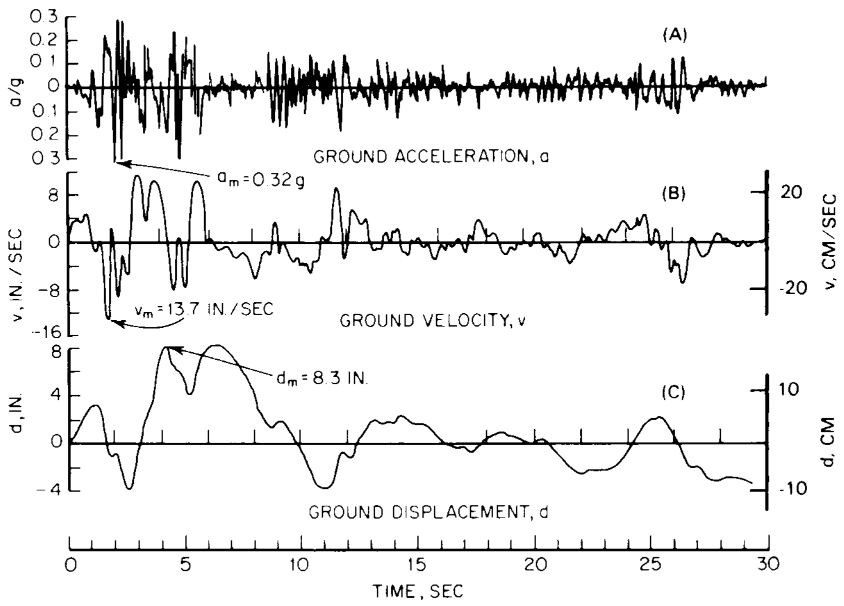

FIGURE 24.5 El Centro, California, earthquake of May 18, 1940, north-south component. (*A*) Record of the ground acceleration. (*B*) Variation of ground velocity v with time, obtained by integration of (*A*). (*C*) Variation of ground displacement with time, obtained by integration of (*B*).

displacement is 8.3 in. (21 cm). These three maximum values are of particular interest because they help to define the response motions of the various structures considered in Fig. 24.3 most accurately if all three maxima are taken into account.

RESPONSE SPECTRA

ELASTIC SYSTEMS

The response of the simple oscillator shown in Fig. 24.4 to any type of ground motion can be readily computed as a function of time. A plot of the maximum values of the response, as a function of frequency or period, is commonly called a *response spectrum* (or *shock response spectrum*). The response spectrum may be defined as the graphical relationship of the maximum response of a single degree-of-freedom linear system to dynamic motions or forces. This concept of a response spectrum is widely used in the study of the response of simple oscillators to transient disturbances; for a number of examples, see Chaps. 8 and 23.

A careful study of Fig. 24.4 will reveal that there are nine quantities represented there: acceleration, velocity, and displacement of the base, mass, and their relative values denoted by u. Commonly the maxima of interest are the maximum deformation of the spring, the maximum spring force, the maximum acceleration of the mass (which is directly related to the spring force when there is no damping), or a quantity having the dimensions of velocity, which provides a measure of the maximum energy absorbed in the spring. The details of various forms of response spectra that can be graphically represented, uses of response spectra, and techniques for computing them are discussed in detail in Refs. 1*b,* 1*c,* and 1*d.* A brief treatment of the applications of response spectra follows. The maximum values of the response are of particular interest. These maxima can be stated in terms of the maximum strain in the spring $u_m = D$, the maximum spring force, the maximum acceleration A of the mass (which is related to the maximum spring force directly when there is no damping), or a quantity, having the dimensions of velocity, which gives a measure of the maximum energy absorbed in the spring. This quantity, designated the pseudo velocity V, is defined in such a way that the energy absorption in the spring is $\frac{1}{2}mV^2$. The relations among the maximum relative displacement of the spring D, the pseudo velocity V, and the pseudo acceleration A, which is a measure of the force in the spring, are

$$V = \omega D \tag{24.4}$$

and

$$A = \omega V = \omega^2 D \tag{24.5}$$

The pseudo velocity V is nearly equal to the maximum relative velocity for systems with moderate or high frequencies but may differ considerably from the maximum relative velocity for very low frequency systems. The pseudo acceleration A is exactly equal to the maximum acceleration for systems with no damping and is not greatly different from the maximum acceleration for systems with moderate amounts of damping, over the whole range of frequencies from very low to very high values.

Typical plots of the response of the system to a base excitation, as a function of period or natural frequency, are called *response spectra* (also called *shock spectra*). Plots for acceleration and for relative displacement, for a system with a moderate amount of damping and subjected to an input similar to that of Fig. 24.5, can be

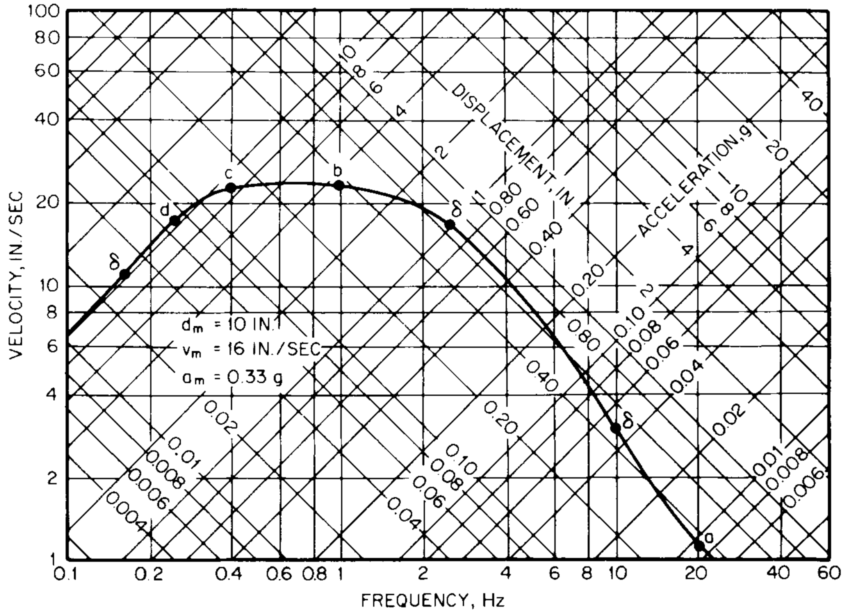

FIGURE 24.6 Smooth response spectrum for typical earthquake.

made. This arithmetic plot of maximum response is simple and convenient to use. Various techniques of computing and plotting spectra may be found in the references cited at the end of this chapter, especially in Refs. 1*c,* 1*d,* and 6 to 18.

A somewhat more useful plot, which indicates the values for D, V, and A, is shown in Fig. 24.6. This plot has the virtue that it also indicates more clearly the extreme or limits of the various parameters defining the response. All parameters are plotted on a logarithmic scale. Since the frequency is the reciprocal of the period, the logarithmic scale for the period would have exactly the same spacing of the points, or in effect the scale for the period would be turned end for end. The pseudo velocity is plotted on a vertical scale. Then on diagonal scales along an axis that extends upward from right to left are plotted values of the displacement, and along an axis that extends upward from left to right the pseudo acceleration is plotted, in such a way that any one point defines for a given frequency the displacement D, the pseudo velocity V, and the pseudo acceleration A. Points are indicated in Fig. 24.6 for the several structures of Fig. 24.3 plotted at their approximate fundamental frequencies. Many other formats are used in plotting spectra; for example, u, $\dot{u}$, ωu, or $\ddot{x}$ vs. time. Such examples are shown in Ref. 1*d.*

Much of the work on spectra, described above, has been developed on the basis of studying strong ground motion categorized by ground motion acceleration level scaling. Another important aspect of statistical study, described in Ref. 19, concerns both ground motions and spectra based on magnitude scaling.

In developing spectral relationships, a wide variety of motions have been considered,[20] ranging from simple pulses of displacement, velocity, or acceleration of the ground, through more complex motions such as those arising from nuclear-blast detonations, and for a variety of earthquakes as taken from available strong-

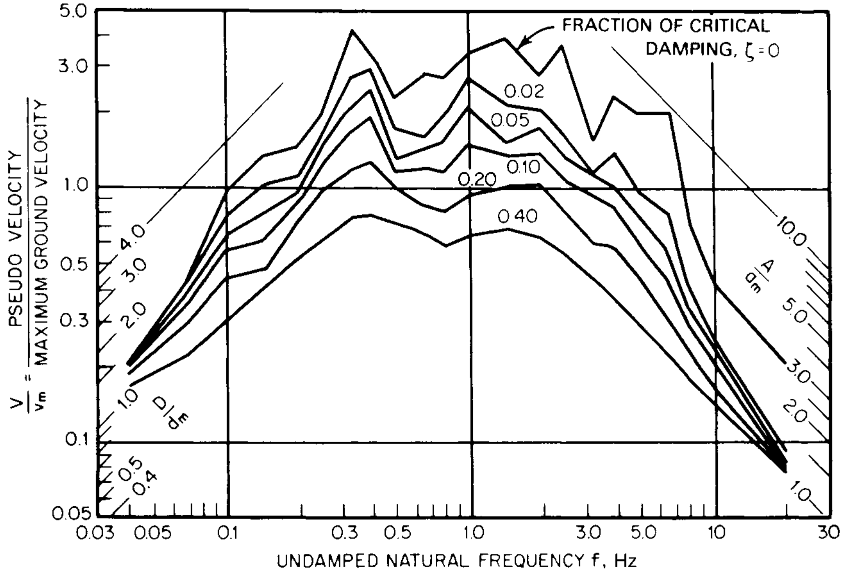

FIGURE 24.7 Response spectra for elastic systems subjected to the El Centro earthquake for various values of fraction of critical damping ζ.

motion records. Response spectra for the El Centro earthquake are shown in Fig. 24.7. The spectrum for small amounts of damping is much more jagged than indicated by Fig. 24.6, but for the higher amounts of damping the response curves are relatively smooth. The scales are chosen in this instance to represent the amplifications of the response relative to the ground-motion values of displacement, velocity, or acceleration.

The spectra shown in Fig. 24.7 are typical of response spectra for nearly all types of ground motion. On the extreme left, corresponding to very low-frequency systems, the response for all degrees of damping approaches an asymptote corresponding to the value of the maximum ground displacement. A low-frequency system corresponds to one having a very heavy mass and a very light spring. When the ground moves relatively rapidly, the mass does not have time to move, and therefore the maximum strain in the spring is precisely equal to the maximum displacement of the ground. For a very high-frequency system, the spring is relatively stiff and the mass very light. Therefore, when the ground moves, the stiff spring forces the mass to move in the same way the ground moves, and the mass therefore must have the same acceleration as the ground at every instant. Hence, the force in the spring is that required to move the mass with the same acceleration as the ground, and the maximum acceleration of the mass is precisely equal to the maximum acceleration of the ground. This is shown by the fact that all the lines on the extreme right-hand side of the figure asymptotically approach the maximum ground-acceleration line.

For intermediate-frequency systems, there is an amplification of the motion. In general, the amplification factor for displacement is less than that for velocity, which in turn is less than that for acceleration. Peak amplification factors for the undamped system ($\zeta = 0$) in Fig. 24.7 are on the order of about 3.5 for displacement, 4.2 for velocity, and 9.5 for acceleration.

The results of similar calculations for other ground motions are quite consistent with those in Fig. 24.7, even for simple motions. The general nature of the response spectrum shown in Fig. 24.8 consists of a central region of amplified response and two limiting regions of response in which for low-frequency systems the response displacement is equal to the maximum ground displacement, and for high-frequency systems the response acceleration is equal to the maximum ground acceleration. Values of the amplification factor reasonable for use in design are presented in the next sections.

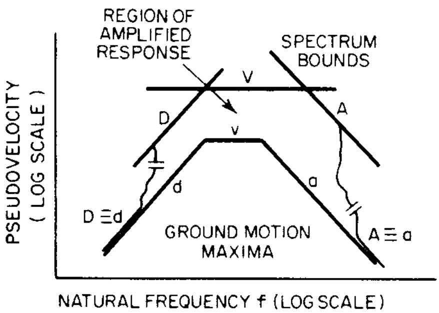

FIGURE 24.8 Typical tripartite logarithmic plot of response-spectrum bounds compared with maximum ground motion.

DESIGN RESPONSE SPECTRA

A response spectrum developed to give design coefficients is called a *design response spectrum* or a *design spectrum.* As an example of its use in seismic design, for any given site, estimates are made of the maximum ground acceleration, maximum ground velocity, and maximum ground displacement. The lines representing these values can be drawn on the tripartite logarithmic chart of which Fig. 24.9 is an example. The heavy lines showing the ground-motion maxima in Fig. 24.9 are drawn for a maximum ground acceleration a of $1.0g$, a velocity v of 48 in./sec (122 cm/sec), and a displacement d of 36 in. (91.5 cm). These data represent motions more intense than those generally considered for any postulated design earthquake hazard. They are, however, approximately in correct proportion for a number of areas of the world, where earthquakes occur either on firm ground, soft rock, or competent sediments of various kinds. For relatively soft sediments, the velocities and displacements might require increases above the values corresponding to the given acceleration as scaled from Fig. 24.9, and for competent rock, the velocity and displacement values would be expected to be somewhat less. More detail can be found in Refs. 1*c* and *d.* It is not likely that maximum ground velocities in excess of 4 to 5 ft/sec (1.2 to 1.5 m/sec) are obtainable under any circumstances.

On the basis of studies of horizontal and vertical directions of excitation for various values of damping,[1c,10,11] representative amplification factors for the 50th and 84.1th percentile levels of horizontal response are presented in Table 24.1. The 84.1th percentile means that one could expect 84.1 percent of the values to fall at or below that particular amplification. With these amplification factors and noting

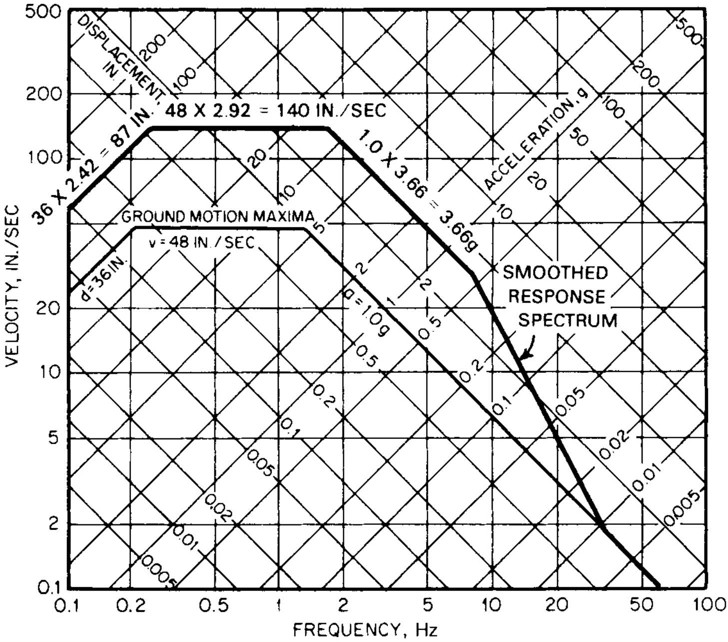

FIGURE 24.9 Basic design spectrum normalized to 1.0*g* for a value of damping equal to 2 percent of critical, 84.1th percentile level. The spectrum bound values are obtained by multiplying the appropriate ground-motion maxima by the corresponding amplification value of Table 24.1.

points *B* and *A* to fall at about 8 and 33 Hz, the spectra may be constructed as shown in Fig. 24.9 by multiplying the ground maxima values of acceleration, velocity, and displacement by the appropriate amplification factors. Further information on, and other approaches to, construction of design spectra may be found in Refs. 1*c* and *d*.

TABLE 24.1 Values of Spectrum Amplification Factors[1c,11]

Percentile	Damping, percent of critical damping	Amplification factor *D*	*V*	*A*
50th	0.5	2.01	2.59	3.68
	2.0	1.63	2.03	2.74
	5.0	1.39	1.65	2.12
	10.0	1.20	1.37	1.64
84.1th	0.5	3.04	3.84	5.10
	2.0	2.42	2.92	3.66
	5.0	2.01	2.30	2.71
	10.0	1.69	1.84	1.99

RESPONSE SPECTRA FOR INELASTIC SYSTEMS

It is convenient to consider an elastoplastic resistance-displacement relation because one can draw response spectra for such a relation in generally the same way as the spectra were drawn for elastic conditions. A simple resistance-displacement relationship for a spring is shown by the light line in Fig. 24.10*A*, where the yield point is indicated, with a curved relationship showing a rise to a maximum resistance and then a decay to a point of maximum useful limit or failure at a displacement u_m; an equivalent elastoplastic resistance curve is shown by the heavy line. A similar elastoplastic resistance function, more indicative of seismic response, is shown in Fig. 24.10*B*. The ductility factor μ is defined as the ratio between the maximum permissible or useful displacement to the yield displacement for the effective curve in both cases.

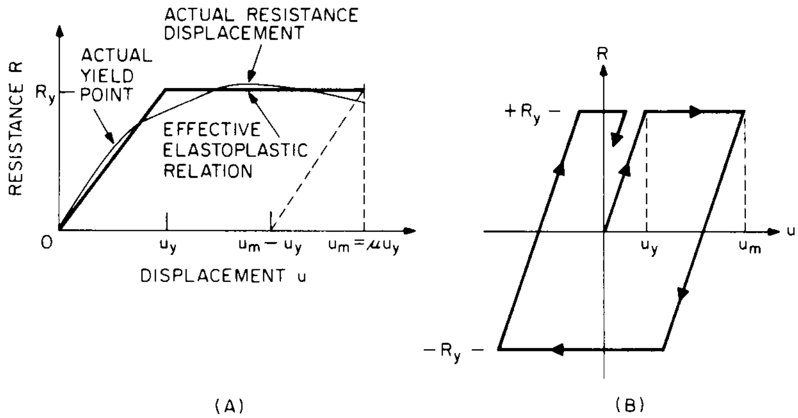

FIGURE 24.10 (*A*) Monotonic resistance-displacement relationships for a spring, shown by the light line; an equivalent elastoplastic resistance curve, shown by the heavy line. (*B*) A similar elastoplastic resistance function, more indicative of seismic response.

The ductility factors for various types of construction depend on the use of the building, the hazard involved in its failure (assumed acceptable risk), the material used, the framing or layout of the structure, and above all on the method of construction and the details of fabrication of joints and connections. A discussion of these topics is given in Refs. 1*c,* 10, and 11. Figure 24.11 shows acceleration spectra for elastoplastic systems having 2 percent of critical damping that were subjected to the El Centro, 1940, earthquake. Here the symbol D_y represents the elastic component of the response displacement, but it is not the total displacement. Hence, the curves also give the elastic component of maximum displacement as well as the maximum acceleration *A,* but they do not give the proper value of maximum pseudo velocity. This is designated by the use of the V' for the pseudo velocity drawn in the figure. The figure is drawn for ductility factors ranging from 1 to 10. A response spectrum for total displacement also can be drawn for the same conditions as for Fig. 24.11. It is obtained by multiplying each curve's ordinates by the value of ductility factor μ shown on that curve.

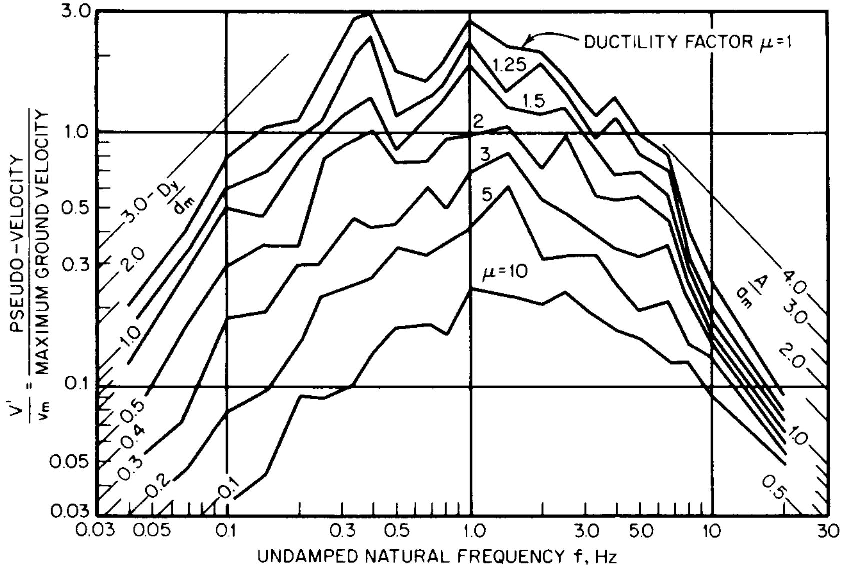

FIGURE 24.11 Deformation spectra for elastoplastic systems with 2 percent of critical damping that were subjected to the El Centro earthquake.

The following considerations are useful in using the design spectrum to approximate inelastic behavior. In the amplified displacement region of the spectra, the left-hand side, and in the amplified velocity region, at the top, the spectrum remains unchanged for total displacement and is divided by the ductility factor to obtain yield displacement or acceleration. The upper right-hand portion sloping down at 45°, or the amplified acceleration region of the spectrum, is relocated for an elastoplastic resistance curve, or for any other resistance curve for actual structural materials, by choosing it at a level which corresponds to the same energy absorption for the elastoplastic curve as for an elastic curve for the same period of vibration. The extreme right-hand portion of the spectrum, where the response is governed by the maximum ground acceleration, remains at the same acceleration level as for the elastic case and, therefore, at a corresponding increased total displacement level. The frequencies at the corners are kept at the same values as in the elastic spectrum. The acceleration transition region of the response spectrum is now drawn also as a straight-line transition from the newly located amplified acceleration line and the ground-acceleration line, using the same frequency points of intersection as in the elastic response spectrum. In all cases the inelastic maximum acceleration spectrum and the inelastic maximum displacement spectrum differ by the factor μ at the same frequencies. The design spectrum so obtained is shown in Fig. 24.12.

The solid line $DVAA_0$ in Fig. 24.12 shows the elastic response spectrum. The heavy circles at the intersections of the various branches show the frequencies which remain constant in the construction of the inelastic design spectrum. The dashed line $D'V'A'A_0$ shows the inelastic acceleration, and the line $DVA''A_0''$ shows the inelastic displacement. These two differ by a constant factor μ for the construction shown, except that A and A' differ by the factor $\sqrt{2\mu - 1}$, since this is the factor that corresponds to constant energy for an elastoplastic resistance.

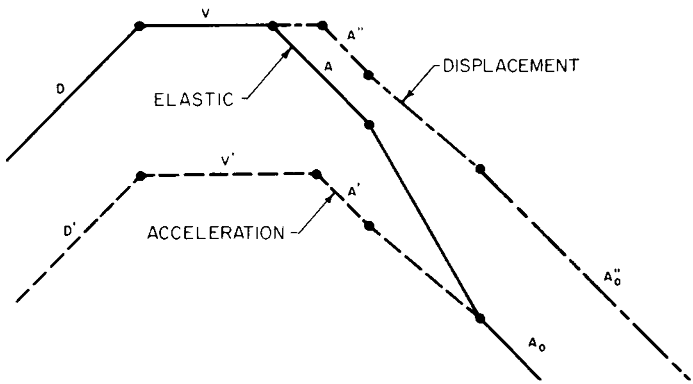

FIGURE 24.12 The normal elastic design spectrum is given by $DVAA_0$. The modified spectrum (see text for rules for construction) representing approximately the acceleration or elastic yield displacement for a nonlinear system with ductility μ is given by $D'V'A''A_0$. The total or maximum displacement for the nonlinear system is given approximately by $DVA''A''_0$ and is obtained by multiplying the modified spectrum by the value μ.

The modified spectrum to account for inelastic action is an approximation at best and should be used generally only for relatively small ductility values, for example, 5 or less. Additional information on the development of elastic and inelastic design response spectra may be found in Refs. 1*c,* 1*d,* and 10 to 21.

MULTIPLE DEGREE-OF-FREEDOM SYSTEMS

USE OF RESPONSE SPECTRA

A multiple degree-of-freedom system has as many modes of vibration as the number of degrees-of-freedom. For example, for the shear beam shown in Fig. 24.13*A* the fundamental mode of lateral oscillation is shown in (*B*), the second mode in (*C*), and the third mode in (*D*). The number of modes in this case is 5. In a system that has independent (uncoupled) modes (this condition is often satisfied for buildings) each mode responds to the base motion as an independent single degree-of-freedom system (see Chap. 21). Thus, the modal responses are nearly independent functions of time. However, the maxima do not necessarily occur at the same time.

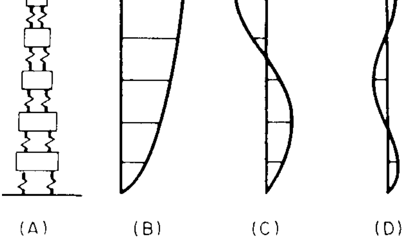

FIGURE 24.13 Modes of vibration of shear beam. The first three (1, 2, 3) relative mode shapes are shown by (*B*), (*C*), and (*D*), respectively, for lateral vibration.

For multiple degree-of-freedom systems, the concept of the response spectrum can also be used in most cases, although the use of the inelastic response spectrum is only approximately valid as a design procedure.[10,11] For a system with a number of masses at nodes in a flexible framework, the equation of motion can be written in matrix form as

$$M\ddot{u} + C\dot{u} + Ku = -M(\ddot{y})\{1\} \tag{24.6}$$

in which the last symbol on the right represents a unit column vector. The mass matrix M is usually diagonal, but in all cases both M and the stiffness matrix K are symmetrical. When the damping matrix C satisfies certain conditions, the simplest of which is when it is a linear combination of M and K, then the system has normal modes of vibration, with modal displacement vectors u_n. Analysis techniques for handling multiple degree-of-freedom systems are described in Ref. 8, as well as Chaps. 21 and 28.

DESIGN

GENERAL CONSIDERATIONS

The design of all types of building structures, as well as the design of building services (such as water, gas, fuel pipelines, water and electrical services, sewage, and vertical transportation) must take into account the effects of earthquakes and wind. (The design of structures for wind loads is covered in Chap. 29, Part II.) Often, these building services are large, expensive, and affect large numbers of people. Thus, the design of a building should consider siting studies to minimize seismic effects or, at very least, identify such effects that must be expected to be accommodated, including faulting; all this must be taken into account, in addition to the usual considerations of functional needs, economics, land acquisition and land use restrictions, transportation, and the availability of labor.

From a design perspective, there must be a rational selection of the applicable loadings (demand)—preferably, examination of the design for a range of loadings, load combinations, and load paths, in order to assess margins of safety—as well as careful attention to modeling and analysis. From the resistance (supply) side, careful attention must be given to the properties of the materials, to connections of structural members and items, as well as to the joining process, to foundations and anchorage, to provisions for controlling ductility and handling transient displacements, to aging considerations, and to the meeting or exceeding applicable code requirements, specifications, and regulations—all in accordance with appropriate professional standards of care and good engineering judgment.

In the design of a building to resist earthquake motions, the designer works within certain constraints, such as the architectural configuration of the building, the foundation conditions, the nature and extent of the hazard should failure or collapse occur, the possibility of an earthquake, the possible intensity of earthquakes in the region, the cost or available capital for construction, and similar factors. There must be some basis for the selection of the strength and the proportions of the building and of the various members in it. The required strength depends on factors such as the intensity of earthquake motions to be expected, the flexibility of the structure, and the ductility or reserve strength of the structure before damage occurs. Because of the interrelations among the flexibility and strength of a structure and the forces generated in it by earthquake motions, the dynamic design procedure must take these various factors into account. The ideal to be achieved is one involving flexibility and energy-absorbing capacity which will permit the earthquake displacements to take place without generating unduly large forces. To achieve this end, careful design (with attention to continuity, redundancy, connections, strength, and ductility), control of the construction procedures, and appropriate inspection practices are necessary. The attainment of the ductility required to resist earthquake motions must be emphasized. If the ductility achieved is less than assumed, then in all likelihood the forces in the structure will be higher than estimated.

The above considerations emphasize the importance of a knowledge of structural behavior and the uncertainties associated therewith, and techniques for assessing and implementing appropriate margins of safety in design. In earthquake engineering design, careful consideration must be given to the cyclic behavior that normally occurs, as opposed to monotonic behavior. Because of this severe cyclic demand on the structural framing and its connections (irrespective of whether or not they are made of reinforced or prestressed concrete or of steel), it is important to consider the strength characteristics of the particular materials and sections as they are joined, including bracing; it is necessary to ensure that the demand for limited ductility can be achieved in a satisfactory manner. Earthquakes throughout the world in the 1990s have shown that certain design assumptions and accompanying fabrication techniques have led to severely decreased strength margins in some cases and/or to serious structural damage. Life safety is the primary matter of concern, but increasingly building owners are more conscious of protecting their plant investment and to preserving production operations without major repair and "down time." Thus the building owner and engineering designer must come to an agreement as to the level of protection desired, based on current knowledge and applicable conditions.

Some typical references for structures, lifelines, and transportation systems (including observation summaries of major earthquakes) are given in Refs. 22 to 36. In addition to these sources, guidelines and regulations are available from associations of manufacturers or major suppliers of steel, concrete, prestressed concrete, masonry, and wood.

EFFECTS OF DESIGN ON BEHAVIOR AND ON ANALYSIS*

A structure designed for very much larger horizontal forces than are ordinarily prescribed will have a shorter period of vibration because of its greater stiffness. The shorter period results in higher spectral accelerations, so that the stiffer structure may attract more horizontal force. Thus, a structure designed for too large a force will not necessarily be safer than a similar structure based on smaller forces. On the other hand, a design based on too small a force makes the structure more flexible and will increase the relative deflections of the floors.

In general, yielding occurs first in the story that is weakest compared with the magnitudes of the shearing forces to be transmitted. In many cases this will be near the base of the structure. If the system is essentially elastoplastic, the forces transmitted through the yielded story cannot exceed the yield shear for that story. Thus, the shears, accelerations, and relative deflections of the portion of the structure above the yielded floor are reduced compared with those for an elastic structure subjected to the same base motion. Consequently, if a structure is designed for a base shear which is less than the maximum value computed for an elastic system, the lowest stories will yield and the shears in the upper stories will be reduced. This means that, with proper provision for energy absorption in the lower stories, a structure

* This section is based partly on material from Ref. 37, by permission, with update modification.

will, in general, have adequate strength, provided the design shearing forces for the upper stories are consistent with the design base shear. Building code recommendations are intended to provide such a consistent set of shears. However, on all levels it is wise to have the energy absorption, if possible, distributed more or less uniformly throughout the structural system, i.e., not concentrated only in a few locations; such a procedure places an unusual, and quite often unbalanced, demand on localized and specific portions of a structure.

A significant inelastic deformation in a structure inhibits the higher modes of oscillation. Therefore, the major deformation is in the mode in which the inelastic deformation predominates, which is usually the fundamental mode. The period of vibration is effectively increased, and in many respects the structure responds almost as a single degree-of-freedom system corresponding to its entire mass supported by the story which becomes inelastic. Therefore, the base shear can be computed for the modified structure, with its fundamental period defining the modified spectrum on which the design should be based. The fundamental period of the modified structure generally will not be materially different from that of the original elastic structure in the case of framed structures. In the case of shear-wall structures it will be longer.

It is partly because of these facts that it is usual in design recommendations to use the frequency of the fundamental mode, without taking direct account of the higher modes. However, it is desirable to consider a shearing-force distribution which accounts for higher-mode excitations of the portion above the plastic region. This is implied in the UBC, SEAOC (Structural Engineers Association of California), and National Earthquake Hazard Reduction Program (NEHRP) recommendations by the provision for lateral-force coefficients which vary with height. The distribution over the height corresponding to an acceleration varying uniformly from zero at the base to a maximum at the top takes into account the fact that local accelerations at higher levels in the structure are greater than those at lower levels, because of the larger motions at the higher elevations, and accounts quite well for the moments and shears in the structure.

Many of the modern seismic analysis approaches are described in detail in Ref. 8. Prevailing analysis techniques employ design spectra or motion time-histories as input. Many benchmarked computer software packages are available that permit fairly sophisticated structural analyses to be undertaken, especially when the modeling is carefully studied and well understood and the input is relatively well defined. Typical of these powerful programs are ETABS, SAP 80, ABAQUS, ANSYS, and ADINA. In the field of soil-structure interaction, computer software packages include SASSI, CLASSI, FLUSH, and SHAKE. Since all such programs are constantly being upgraded, it is necessary to keep abreast of such modifications.

In the case of intense earthquakes, the ensuing ground motions can be of the sharp, impulsive type. When such ground motions impinge on a structure, the effect is literally that of a shock. Moreover, the impulses can be multiple in nature, so that if the timing between impulses is quite short, the rapid shock-type motion transmitted to building frames may be intensified. Such an intense form of impulsive input has been observed in earthquakes in Northridge, California and in Kobe, Japan; it may lead to serious structural problems in buildings if such input has not been properly considered in the building's design and construction. Although not explicitly spelled out in present building codes, it is expected that a strength check would be carried out to see that the gross building shearing resistance is sufficient (including normal margins of strength) to resist an intense shock characterized by the zero period acceleration (ZPA); in addition, structural members must have ample tensile and compressive resistance so that they are able to resist a vertical or oblique type of shock. This intense type of input subsequently leads to the vibratory type of motion that is commonly treated in seismic analysis. Fortunately, in most earthquakes, the initial motions that lead to building vibration are small enough to be accommodated by the resistance of most buildings.

The strength checks, referred to above, have nothing to do with the principal modes of vibration of a building as determined by analysis; in reality, the structure or piece of equipment is initially at rest; then it must respond in a quasi-rigid mode to these intense impulses. In that sense the entire mass of the building is active in providing resistance. The forces under those circumstances can be quite high. However, in some cases where the design calls for the lateral and vertical forces to be carried in just a few frames or members, the imparted forces can be immense. Fortunately, most buildings have ample resistance to accommodate such effects—especially if the base anchorage and connections are well constructed for a requisite set of structural frames. Similarly, most equipment that is properly mounted has more than enough margin of strength to accommodate the imposed intense dynamic loading. Analysis of earthquake damage, with regard to difficulties with connections and details in both steel and concrete structures, suggests that adequate attention is required in the design of details, in the quality of their fabrication, and in the quality of their construction in order to assure their adequate performance. In this respect, Ref. 36 concerned with the quality of construction is pertinent.

As a result of the damage experienced in the 1989 Loma Prieta earthquake, the 1994 Northridge earthquake, and the 1995 Kobe earthquake, numerous studies have been made of the performance of structural building forms and elements, especially connections. At the same time, building codes are rapidly undergoing major revisions. One of the largest R&D studies was conducted on steel moment-frame buildings,[37] which is leading to changes in the provisions of the AISC steel provisions.[38] At the same time, many revisions have occurred in the provisions for reinforced concrete[39] and, in the case of prestressed concrete structures, one needs to keep abreast of the developments reported in the 1999 and later *PCI Journal.* Engineers and architects involved in the design of steel and concrete structures are advised to keep abreast of the latest technical literature in the fields sited.

DESIGN LATERAL FORCES

Although the complete response of multiple degree-of-freedom systems subjected to earthquake motions can be calculated (see Chap. 28, Part II), it should not be inferred that it is generally necessary to make such calculations as a routine matter in the design of multistory buildings. There are a great many uncertainties about the input motions and about the structural characteristics that can affect the computations. Moreover, it is not generally necessary or desirable to design tall structures to remain completely elastic under severe earthquake motions, and considerations of inelastic behavior lead to further discrepancies between the results of routine methods of calculation and the actual response of structures.

The Uniform Building Code[25] recommendations, with proper attention to the *R* and *S* values, for earthquake lateral forces are, in general, consistent with the forces and displacements determined by more elaborate procedures. A structure designed according to these recommendations will remain elastic, or nearly so, under moderate earthquakes of frequent occurrence, but it must be able to yield locally without serious consequences if it is to resist a major earthquake. Thus, design for the required ductility is an important consideration.

The ductility of the material itself is not a direct indication of the ductility of the structure. Laboratory and field tests, and data from operational use of military weapons tests indicate that structures of practical configurations having frames of ductile materials, or a combination of ductile materials, exhibit ductility factors μ ranging from a minimum of 3 to a maximum of 8. For a quality constructed structure with well-distributed energy absorption, a ductility factor of about 3 to 5, or even less, for critical facilities is a reasonable criterion when designed to IBC earthquake requirements.

As a result of the numerous earthquakes that have occurred throughout the world and of the resulting loss of life and property, seismic design codes have undergone major revisions to reflect a modern understanding of dynamic design, based on research, and to reflect lessons learned in recent damaging earthquakes. Building codes, with their applicable provisions, are undergoing rapid and major revisions. A major advance has occurred with the issuance of an international building code.[40] Other relatively recent structural provision changes are reflected in the Uniform Building Code[25] and the NEHRP,[27] with much of the latter material subsumed into the International Building Code.[40] At the same time, major changes in other codes and specifications are being made, as described earlier herein.

The complexity of any such modern code requires that the provisions, along with the commentary, be studied in detail prior to performing detailed computations. In general the seismic coefficients have been increased in comparison to earlier values, and the approaches being adopted attempt to take more factors into consideration in arriving at the design base shear.

SEISMIC FORCES FOR OVERTURNING MOMENT AND SHEAR DISTRIBUTION

In general when modal analysis techniques are not used, in a complex structure or in one having several degrees-of-freedom, it is necessary to have a method of defining the seismic design forces at each mass point of the structure in order to be able to compute the shears and moments to be used for design throughout the structure. The method described in the SEAOC, UBC, IBC, or NEHRP provisions is preferable for this purpose. Obviously, the proper foundations, and adequate anchorage, are required.

DAMPING

The damping in structural elements and components and in supports and foundations of the structure is a function of the intensity of motion and of the stress or strain levels introduced within the structural component or structure, and is highly dependent on the makeup of the structure and the energy absorption mechanisms within it. For further details see Refs. 1 and 12.

GRAVITY LOADS

The effect of gravity loads, when the structures deform laterally by a considerable amount, can be of importance. In accordance with the general recommendations of most extant codes, the effects of gravity loads are to be added directly to the primary and earthquake effects. In general, in computing the effect of gravity loads, one must take into account the actual deflection of the structure, not the deflection corresponding to reduced seismic coefficients.

VERTICAL AND HORIZONTAL EXCITATION

Usually the stresses or strains at a particular point are affected primarily by the earthquake motions in only one direction; the second direction produces little if any influence. However, this is not always the case and is certainly not so for a simple square building supported on four columns where the stress in a corner column is in general affected equally by the earthquakes in the two horizontal directions, and may be affected also by the vertical earthquake forces. Since the ground moves in all three directions in an earthquake, and even tilts and rotates, consideration of the combined effects of all these motions must be included in the design. When the response in the various directions may be considered to be uncoupled, consideration can be given separately to the various components of base motion, and individual response spectra can be determined for each component of direction or of transient base displacement. Calculations have been made for the elastic response spectra in all directions for a number of earthquakes. Studies indicate that the vertical response spectrum is about two-thirds the horizontal response spectrum, and it is recommended that a ratio of 2:3 for vertical response compared with horizontal response be used in design. If there are systems or elements that are particularly sensitive to vertical shock, these will require special design consideration.

For parts of structures or components that are affected by motions in various directions in general, the response may be computed by either one of two methods. The first method involves computing the response for each of the directions independently and then taking the square root of the sums of the squares of the resulting stresses in the particular direction at a particular point as a combined response. Alternatively, one can use the second method of taking the seismic forces corresponding to 100 percent of the motion in one direction combined with 40 percent of the motions in the other two orthogonal directions, adding the absolute values of the effects of these to obtain the maximum resultant forces in a member or at a point in a particular direction, and computing the stresses corresponding to the combined effects. In general, this alternative method is slightly conservative. A related matter that merits attention in design is the provision for relative motion of parts or elements having supports at different locations.

UNSYMMETRICAL STRUCTURES IN TORSION

In design, consideration should be given to the effects of torsion on unsymmetrical structures and even on symmetrical structures where torsions may arise from off-center loads and accidentally because of various reasons, including lack of homogeneity of structures or the presence of the wave motions developed in earthquakes. Most modern codes provide values of computed and accidental eccentricity to use in design, but in the event that analyses indicate values greater than those recommended by the code, the analytical values should be used in design.

SIMULATION TESTING

Simulation testing to create various vibration environments has been employed for years in connection with the development of equipment that must withstand vibration. Over the years such testing of small components has been accomplished on shake tables (see Chap. 25) and involves many different types of input functions. As a result of improved development of electromechanical rams, large shake tables have been developed which can simulate the excitation that may be experienced in a building, structural component, or items of equipment, from various types of ground motions, including earthquake motions, nuclear ground motions, nuclear blast motions induced in the ground or in a structure, and traffic vibrations. Some of these devices are able to provide simultaneous motion in three orthogonal directions. For larger items analysis may be the tool available for assessment of adequacy, coupled with physical observation during transport.

The matter of simulation testing became of great importance with regard to earthquake excitation because of the development of nuclear power plants and the necessity for components in these plants to remain operational for purposes of safe shutdown and containment, and also because of the observed loss of lifeline items in recent earthquakes as, for example, communication and control equipment, utilities, and fire-fighting systems. It is common to require computation of floor response spectra[21] and to provide for equipment qualification.

EQUIPMENT AND LIFELINES

No introduction to earthquake engineering would be complete without mention of the importance of adequate design of equipment in buildings and essential building services, including, for example, communications, water, sewage and transportation systems, gas and liquid fuel pipelines, and other critical facilities. Design approaches for these important elements of constructed facilities, as well as sources of energy, have received major design attention in recent years as the importance of maintaining their integrity has become increasingly apparent.

It has always been obvious that the seismic design of equipment was important, but the focus on nuclear power has pushed this technology to the forefront. Many standards and documents are devoted to the design of such equipment. As a starting point for gaining information about such matters, the reader is referred to Refs. 34 through 36 and 41 through 43. Design considerations for critical industrial facilities, meaning those industries that require less attention than a nuclear power plant, but more than a routine building, are discussed in Ref. 44.

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